Study Conducted for the

MISSISSIPPI STATE OFFICE
SOIL CONSERVATION SERVICE
U.S. DEPARTMENT OF AGRICULTURE

by the

North Central Region
Agricultural Research Service
U.S. DEPARTMENT OF AGRICULTURE

in cooperation with the

MINNESOTA AGRICULTURAL EXPERIMENT STATION

and the

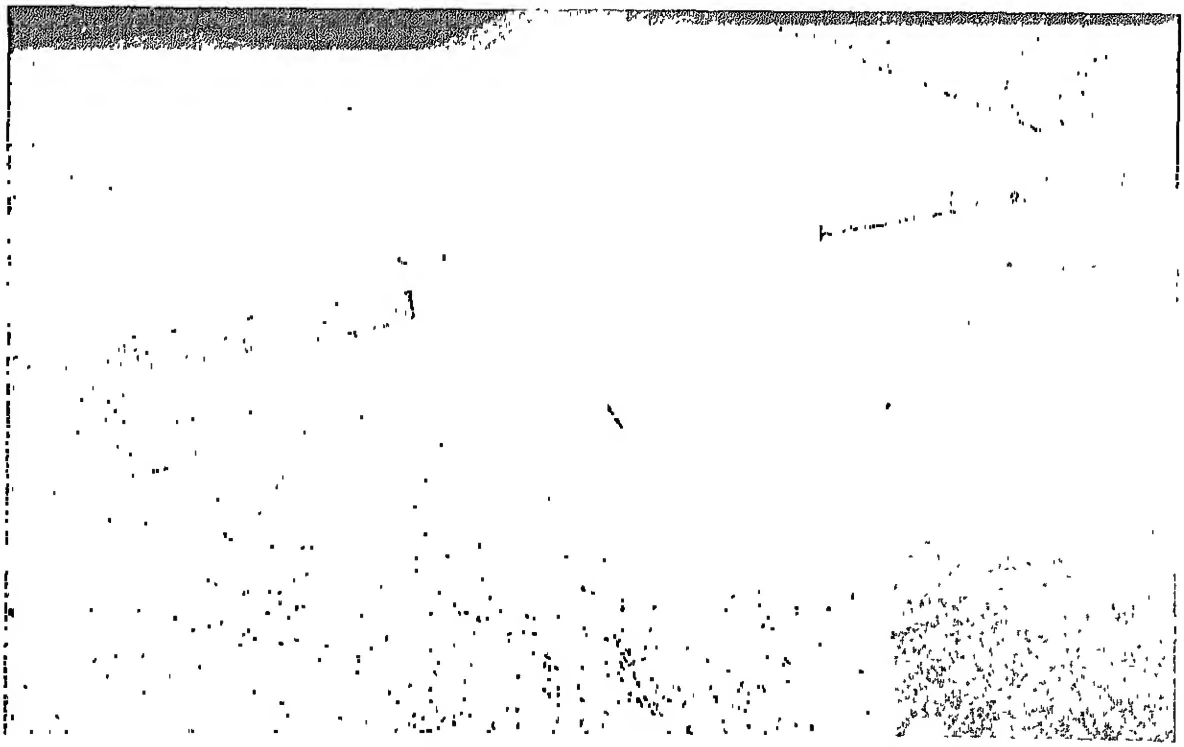
ST. ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

PREFACE

The study reported here, authorized in a Letter of Agreement between representatives of the Agricultural Research Service and the Soil Conservation Service, covers the construction of a large model of a box inlet drop spillway, tests to determine the need for protective revetment at the inlet and outlet of the structure, and an analysis and report of the observations. The Letter of Agreement was subsequently modified by correspondence and consultation to include the development of a spillway crest, stilling basin wingwall modifications, and a downstream channel riprap plan that would better fit site conditions.

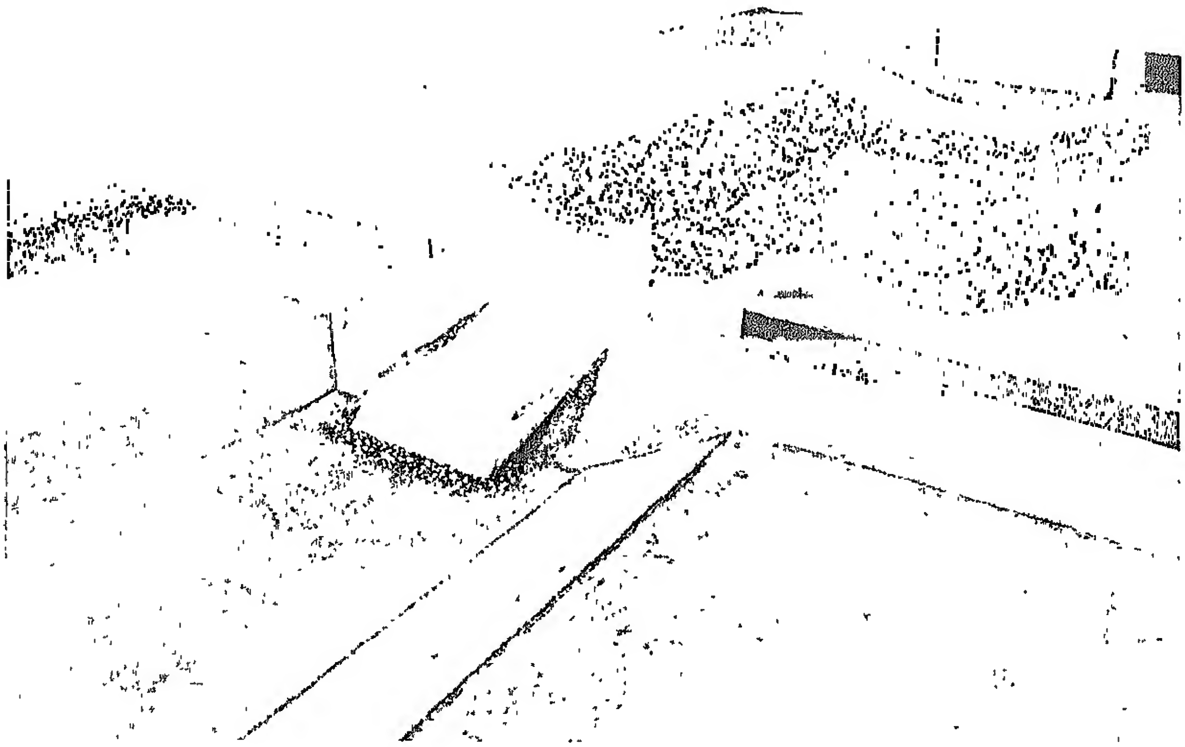
The Letter of Agreement was initiated on December 29, 1969, by W. L. Heard, State Conservationist, Soil Conservation Service, Jackson, Miss.; recommended on January 8, 1970, by A. R. Robinson, Director of the USDA Sedimentation Laboratory, Agricultural Research Service, Oxford, Miss.; further recommended on January 12, 1970, by Fred W. Blaisdell, Hydraulic Engineer, Agricultural Research Service, St. Anthony Falls Hydraulic Laboratory, Minneapolis, Minn.; and approved on January 15, 1970, by C. H. Wadleigh, Director of the Soil and Water Conservation Research Division, Agricultural Research Service, Beltsville, Md.

Basic design data were furnished by and liaison during the study maintained through A. C. Allnutt, Assistant State Conservation Engineer, and K. M. Hayward, Head, State Design Unit, Soil Conservation Service, Mississippi. For the Agricultural Research Service, the study was conducted by Hydraulic Engineers Charles A. Donnelly, Kesavarao Yalamanchili, and Clayton L. Anderson under the direction of Fred W. Blaisdell.



Initial Design

Recommended Design



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MODEL TEST OF BOX INLET DROP SPILLWAY AND STILLING BASIN PROPOSED FOR TILLATOBA CREEK, TALLAHATCHIE COUNTY, MISS.

By Fred W. Blaisdell, hydraulic engineer, North Central Region, Agricultural Research Service

ABSTRACT

Reported here for a box inlet drop spillway proposed for Tillatoba Creek, Tallahatchie County, Miss., are the experimental design of the riprap in the approach and exit channels and the development of a spillway crest to fit a predetermined head-discharge relationship.

Excessive scour in the approach channel near the structure was eliminated by adding riprap protection. The areas to be protected by each riprap size were determined from isovels measured in the approach channel. The sizes of riprap required to resist movement caused by the accelerating velocities approaching the structure were based on the Manning formula. In the Manning formula the bed roughness was related to the mean riprap size, and the slope of the hydraulic grade line was evaluated from the critical tractive force using the criteria developed by A. G. Anderson et al., St. Anthony Falls Hydraulic Laboratory, University of Minnesota.

The dropdown of the overbank flow into the approach channel near the structure and the resulting erosion of the approach channel banks were eliminated by designing the box inlet crest so the head on the crest at bank-full discharge was equal to the depth of flow. The developed crest shape is that of a compound trapezoidal weir. Equations for the compound trapezoidal weir head-discharge relationship were derived. The discharge coefficients in the equations were determined and checked against the observations. The agreement was within 6 percent.

The self-formed shape of the sand-bed downstream channel was determined. Site requirements necessitated modifications of this shape. Experimental design methods were used to determine the placement and sizes of riprap required to protect the downstream channel.

The effect of tailwater depths less than the design depth was shown to be minimal.

INTRODUCTION

Project Location

Tillatoba Creek, a tributary of the Mississippi-Yazoo-Tallahatchie River system, is located in northwestern Mississippi. The watershed lies in Tallahatchie, Yalobusha, and the very northwest corner of Grenada Counties. The proposed grade stabilization structure on which tests were made will be located at station 58 + 00 ± on main Tillatoba Creek. It is the largest of four proposed similar structures. The others will be located at station 136 + 50 on the main creek and at stations 96 + 20 and 150 + 00 on the South Fork of Tillatoba Creek.

Background

At the locations of the proposed structures the channel of Tillatoba Creek is incised into the relatively wide valley floor and is actively degrading. Drop structures will be required to stabilize the grade. The soil is fine and easily erodible. Riprap will be required to prevent scour in the vicinity of the structure, but the placement and size of the riprap were unknown. Since no riprap was available locally and the riprap shipped in from Alabama was costly, it was essential that the quantities required be kept as small as possible.

The channel will be improved in the vicinity of the structures. Even after improvement, the channel, which is designed for an annual flow, will not have sufficient capacity to carry the maximum flood flows, so part of the spillway design discharge will flow in the overbank area and will return to the channel near the spillway. If the structure capacity exceeds the channel capacity, there will be a drawdown of the water surface near the structure. This means that the overbank flow returning to the channel will have to drop into the channel and that the channel banks may be eroded. In order to eliminate the drop into the channel at the structure it would be desirable to have the stage-discharge relationship for the structure provide a bank-full channel at the structure for bank-full channel capacity. The requirement of a stage-discharge relationship for the structure that would fit a predetermined stage-discharge relationship for the site necessitated the development of a nonstandard crest for the box inlet drop spillway grade stabilization structure.

Although computations had been made regarding the tailwater depth at the structure, the active erosion of the channel cast some doubt on whether the tailwater depth-discharge relationship was stable. Tests would be required to determine the effect of a possible lowering of the tailwater depth on the stilling basin performance.

Preliminary tests on a 1/80 size model of a proposed box inlet drop spillway for Tillatoba Creek were made by Neil L. Coleman, geologist, Agricultural Research Service, at the USDA Sedimentation Laboratory, Oxford, Miss. Dr. Coleman concluded:

> These model tests indicate that extensive areas of pronounced channel bed and bank damage could be expected if the box-inlet drop spillway is installed in unprotected channels.
>
> . . . , the entire upstream face of the cross-dike would require protection. The bed and banks of the channel would also need protection for a distance of at least 80 feet upstream from the weir. Bed and bank protection would also have to be extended at least 140 feet below the end sill, and some sort of bank protection would have to be provided beyond that, to control channel widening.
>
> If for some reason high flow with an abnormally low tailwater were to occur, extreme conditions would exist, and any contemplated protective measures would have to be designed to meet these conditions.

This background information indicates the necessity of making model tests to experimentally design the structure.

Purpose and Scope of the Model Study

The Letter of Agreement between the Soil Conservation Service (SCS) and the Agricultural Research Service (ARS) specified that the tests of a large model determine the extent of scour and the need for protection by revetment at the inlet and outlet of the structure.

During the discussions preceding the laboratory tests, it was agreed that the area requiring riprap protection, the shape of the riprapped area, and the minimum size of riprap would be determined. The actual size of the riprap, which would include a safety factor, would be specified by the SCS. The tests would be conducted using the design discharge and tailwater elevations. In addition, the tailwater would be lowered to determine the "sweep-out" or tailwater elevation at which the performance of the stilling basin became poor and the effect of this poor performance.

During these preliminary discussions, which were held at Oxford, Miss., on January 21, 1970, and at Memphis, Tenn., on January 29, 1970, C. T. Myers and K. M. Hayward pointed out to F. W. Blaisdell the desirability of "choking" the structure crest to insure a bank-full head on the structure crest at bank-full structure flow. No agreement was reached regarding development of a choked crest, but the desirability of such a crest was emphasized and recognized.

As a result of these considerations and the results obtained during the tests, the model study determined—

(1) The design of the "choked" crest.
(2) Modifications of the stilling basin wingwalls.
(3) For the approach channel:
  (a) The extent of potential scour in the vicinity of the spillway crest.
  (b) The locations and sizes of riprap required to protect the model channel.
  (c) The velocity distribution (required to determine the riprap sizes).
  (d) Procedures for designing the prototype riprap.
  (e) A test of the riprap design procedure.
(4) For the downstream channel:
  (a) The scour pattern and extent of scour with a sand bed.
  (b) The design of the initial riprap placement and size.
  (c) The performance of the initial riprap design.
  (d) A redesign of the riprap.
  (e) The performance of the redesigned riprap size and placement.
(5) The effect of low tailwater on the structure performance.
(6) For the choked crest (a compound trapezoidal weir):
  (a) The head-discharge relationship.
  (b) Equations for determining the head-discharge relationship.

The test results are presented and analyzed in this report. It is anticipated that the approach channel riprap design procedure and the compound trapezoidal weir head-discharge information will be of general quantitative value, whereas the approach channel velocity distribution, the downstream channel shape, and the downstream channel riprap placement will likely have general qualitative value.

THE PROTOTYPE

The prototype spillway is shown in figure 1. It was designed by C. T. Myers in January 1970 from basic information furnished by A. C. Allnutt.

The spillway is designed for a discharge of 11,200 cubic feet per second (c.f.s.), 80 percent of the peak design runoff of 14,000 c.f.s., because valley storage is available to reduce the peak flow rate at the structure. The channel design capacity is 4,000 c.f.s. Additional specified basic site information, in feet, is: Approach channel elevation 204.0, approach channel width 80, outlet channel elevation 192.0, outlet channel width 60, tailwater elevation 209.0, and structure freeboard elevation 222.0.

The structure proportions developed by C. T. Myers are shown in figure 1, with modifications to the wingwall and dam fill at the stilling

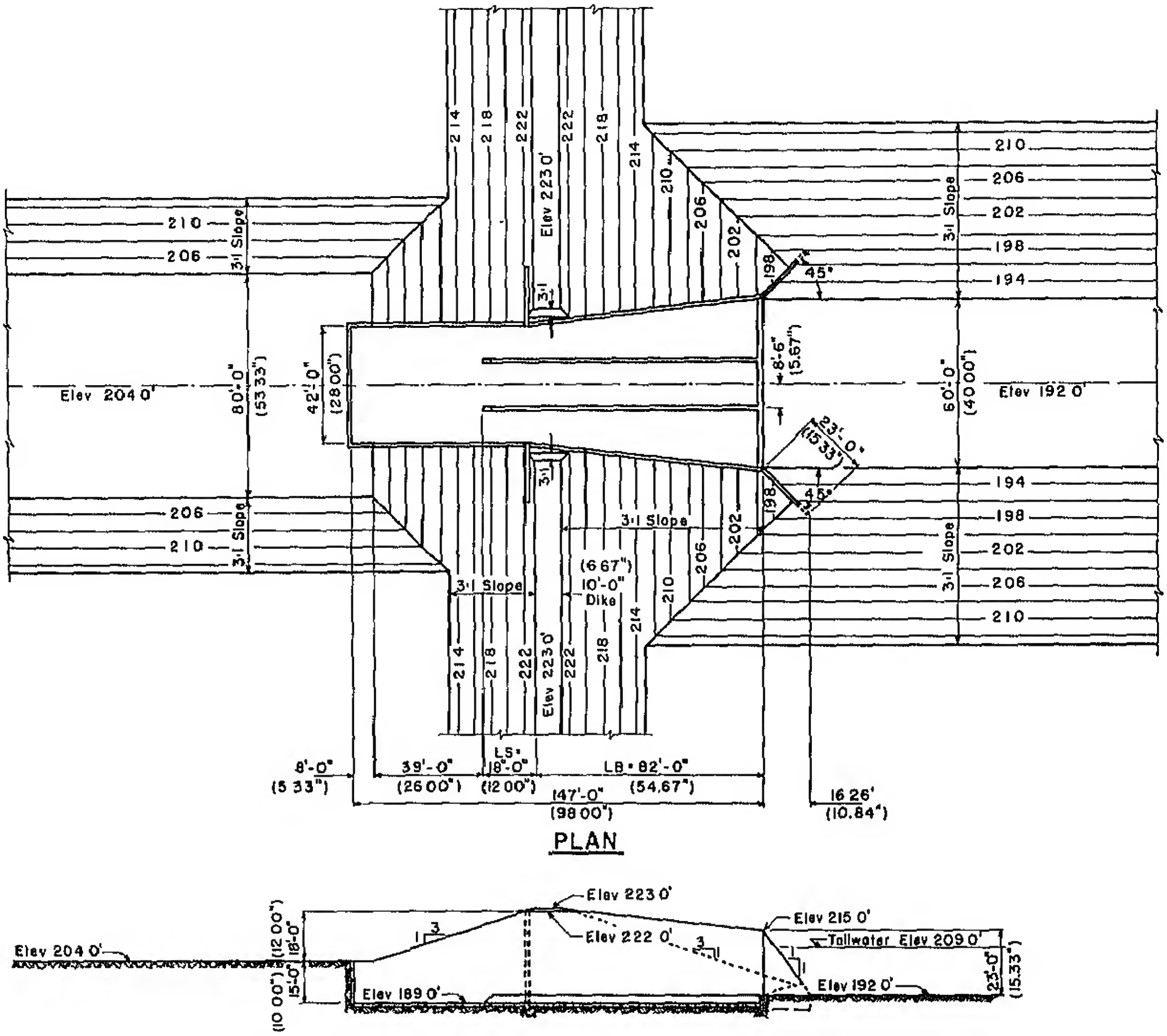


Figure 1.—Initial spillway proportions.

basin exit developed by correspondence and telephone consultation.

The rating curve developed by Myers is shown in figure 2. Also shown as crosses (X) are the bank-full and the design capacity heads and discharges—points through which it would be desirable to have the rating curve pass if the structure proportions could be modified to achieve this preferred rating.

A valley cross section furnished by Myers shows a maximum variation in level from elevation 213 feet[1] of about +1.2 to −2.7 feet over a width of 1,065 feet.

THE MODEL

The model was constructed to the largest practical scale in the available space (a channel 22 ft. 4 in. wide by 45 ft. 9 in. long by 4 ft. 0 in. deep) and with the readily available water supply (11.0 c.f.s. maximum). The model-prototype scale ratio selected was 1:18.

[1] Throughout this report, elevation is in feet.

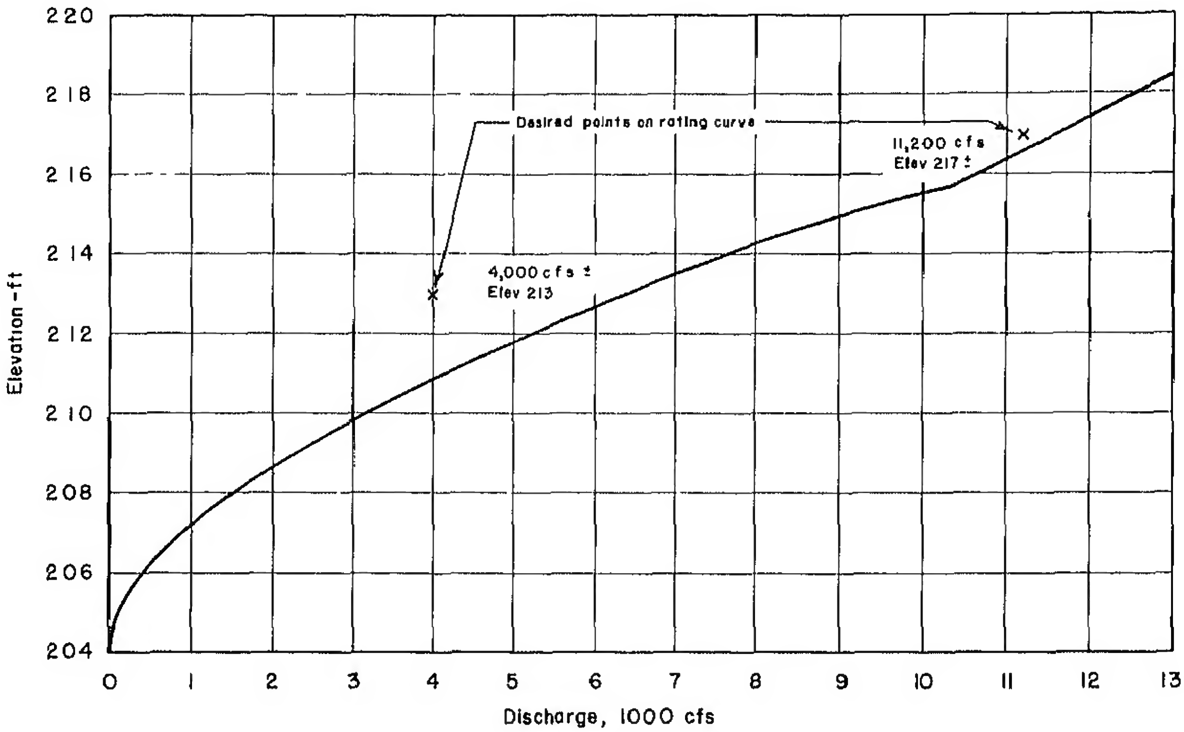


Figure 2.—Initial spillway rating curve.

Description of Model

The dimensions of the model are shown in parentheses in figure 1. Pictures of the model are shown in figure 3.

The walls and floor of the model were made of 3/4-inch plywood. This corresponds to a prototype wall and crest thickness of 13½ inches. The corners of the crest were square.

A watertight steel wall divided the test channel into upstream and downstream sections. The base of the wall, 1.0 foot high, was a steel channel extending across the test channel. Steel panels 2 feet 6 inches high by 8 feet 3 inches long extended from each channel wall on top of the base channel. They projected through the dike as shown in figure 3. The model was located between the panels, and the bulkhead was completed with plywood to prevent seepage through the sand dike.

The upstream and downstream channels were formed in coarse concrete sand. The sand was submerged and was compacted with a concrete vibrator to a uniform density. White yarn was placed along the edges of the plane surfaces in figure 3 to emphasize the channel bed and banks and the dike configurations.

Appurtenances

Water was supplied to the model through the 12-inch pipe shown at the top left in figure 3, A. The water was measured by the drop in pressure across either an 8-inch orifice in the 12-inch line or a 3-inch orifice in the 6-inch bypass line. The orifices had been calibrated for prior studies but were checked before the Tillatoba model was installed. The indicated precision was ± 2 percent. After passing through gate valves, the flow entered the channel through a cone.

Shown at the base of the cone in figure 3, A, are baffles to distribute the flow. They proved insufficient. After test 1 additional baffles were installed to distribute the flow and wire mesh was added to control the turbulence. The flow distribution was checked with a pygmy current meter and the baffles were shifted to achieve a satisfactory flow distribution.

Even with the baffling the approach channel

was too short to give a desirable normal flow distribution at the structure. To achieve better flow distribution a smaller model would have been required. It was decided that the best alternative in this case was to tolerate the short approach channel in order to better evaluate the scour and scour protection.

Figure 3, B, shows the downstream end of the model. A wood bulkhead, cut to the shape of the channel and the overbank, retains the sand. A sand trap is located between the bulkhead and the channel endwall.

The adjustable gate in the downstream endwall shown in figure 3, B, has an effective length of 6 feet 5 inches. The gate controls the tailwater elevation.

A movable bridge supported on rails, which were mounted on the channel sidewalls, spanned the entire test channel. There were two parts to this bridge—a walkway for personnel and a rail to support a point gage.

The rails were carefully leveled against a still water surface. The precision of the point gage readings was about ± 0.002 foot.

Scale Relations

The relation between model and prototype quantities is based on the Froude model law. The multiplication factors to obtain prototype properties from model properties are:

Dimension	Ratio	Multiplication factor
Length	18^1	18
Area	18^2	324
Volume	18^3	5,832
Velocity	$18^{1/2}$	4.24
Time	$18^{1/2}$	4.24
Discharge	$18^{5/2}$	1,375
Weight	18^3	5,832

Because the riprap is relatively large, the indicated riprap sizes in the downstream channel can be quantitatively transferred to the prototype. However, because the bed material in Tillatoba Creek is smaller than the bed material used in the model, the scour dimensions indicate only qualitatively the anticipated prototype scour.

Bed Material

Analyses of the sand bed and the various sizes of riprap used were made to determine the median size, the maximum size, and the size distribution as indicated by the standard deviation σ. The results of these analyses are given in figure 4 and are summarized in table 1.

The sand used for the channel bed is classed as "coarse sand." The sand in Tillatoba Creek is

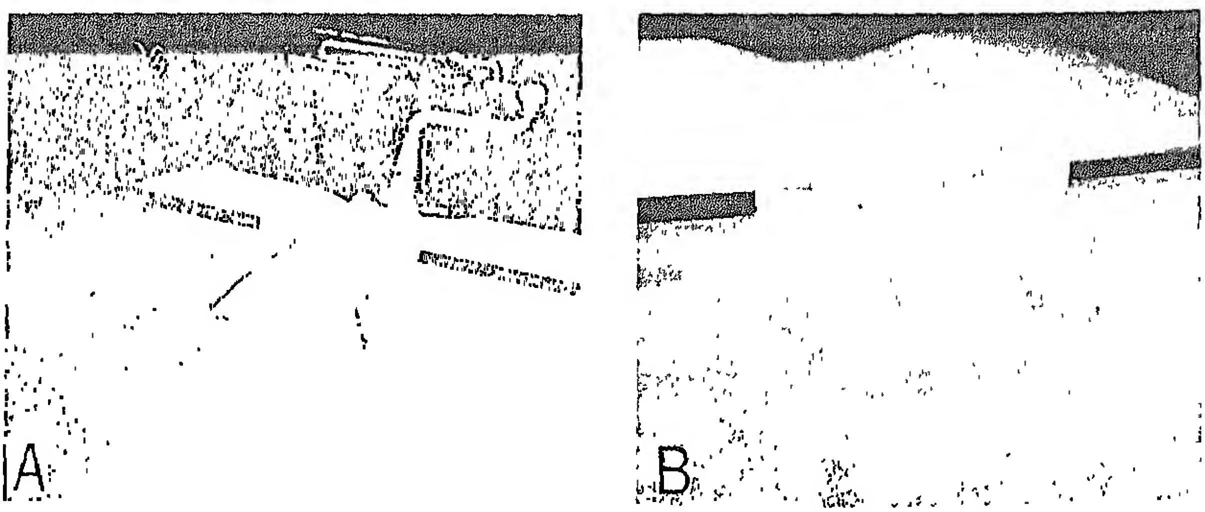


Figure 3.—Model prior to tests: A, Looking upstream; B, looking downstream.

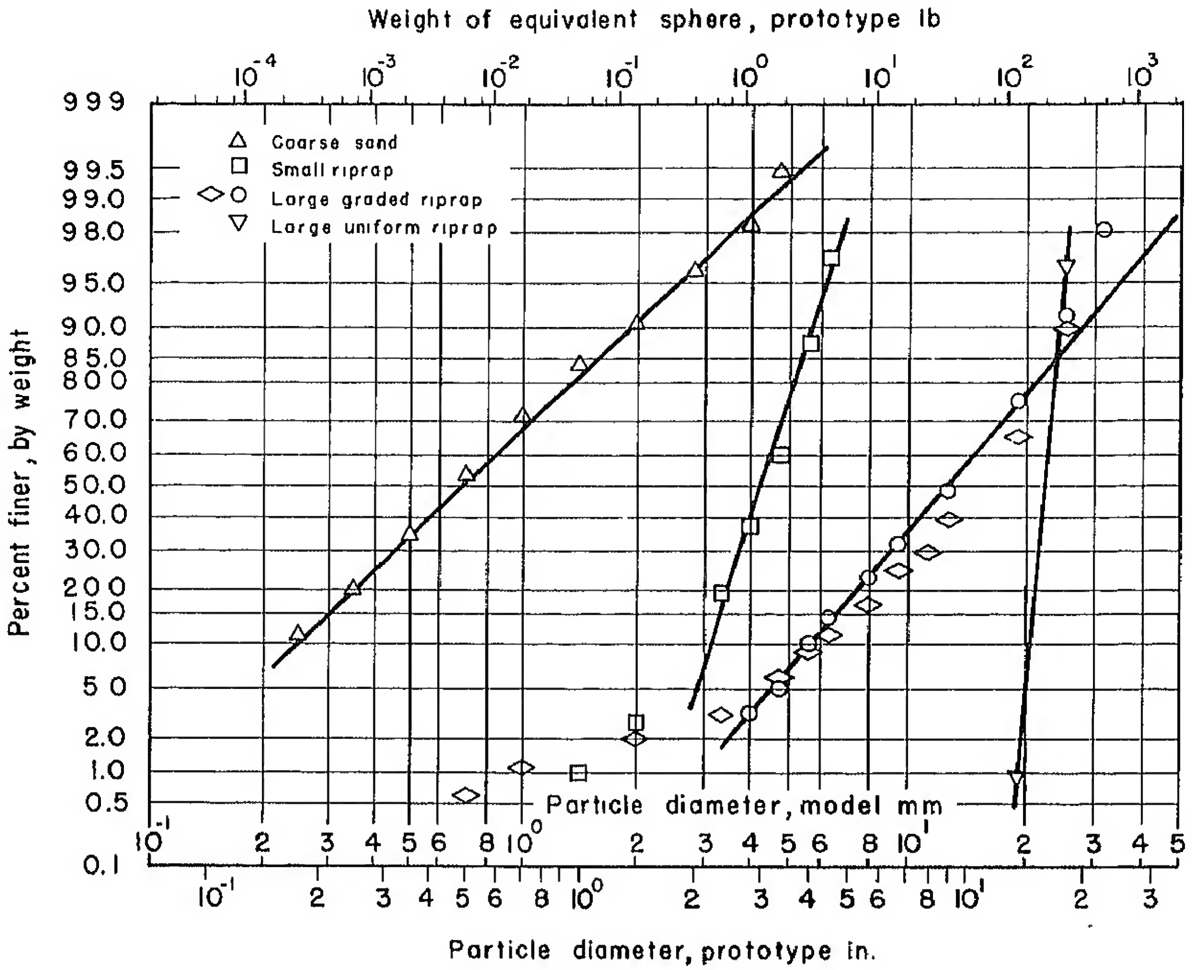


Figure 4.—Analyses of bed material.

so fine that there was no possibility of using a scaled-down size in the model; similarity of movement could not have been obtained. So concrete sand, actually coarser than the Tillatoba Creek bed material, was used in the model. The coarse sand would, however, scour in the model to give qualitative information regarding the magnitude of the scour problem.

The small riprap was a crushed material of roughly uniform size that had been used in a previous model.

The large graded riprap was also available from previous studies. It was partially rounded and partially crushed. This riprap was used in locations where the small riprap was inadequate to prevent scour.

The large uniform riprap was of the same maximum size as the large graded riprap. It was used in areas where the graded riprap scoured until it had been armored by its largest particles.

The large uniform riprap was used with a filter blanket of large graded riprap to meet the filter size requirements. (The criteria listed in table 1 indicate that small riprap should have been used between the large graded and large uniform riprap, or substituted for the large graded riprap. However, there was no apparent serious movement of the large graded riprap fines through the large uniform riprap.) As shown in table 1 no filter is indicated nor was any used between the coarse sand and either the small or large graded riprap.

Filters are sometimes required to prevent leaching of fines in the base material through the protective riprap. The filter criteria used

Table 1.—Analyses of bed materials

Properties

Item		Coarse sand	Small riprap	Large graded riprap	Large uniform riprap
Median size-(D_{50}).					
Model	mm	0.70	4.25	12.7	22.5
Prototype	in	.50	3.0	9.0	15.9
Do	lb	.01	1.3	36	201
Maximum size:					
Model	mm	5	8	35	32
Prototype	in	3.5	5.7	25	23
Do	lb	2.1	9.3	781	608
σ		2.3	1.3	2.0	1.1

Evaluation of properties

Item	$\frac{D_{15}\text{ filter}}{D_{85}\text{ base}}$	$\frac{D_{15}\text{ filter}}{D_{15}\text{ base}}$
Davis-Sorensen criteria	< 4 to 5	> 4 to 5
USBR criteria	< 5	5 to 40
Base - - Filter		
Coarse sand - - Small riprap	2.0 ok	11 ok
Coarse sand - - Large graded	3.9 ok	21 ok
Coarse sand - - Large uniform	13 ng[1]	70 ok to ?
Small riprap - - Large graded	5.4 ng	2 ng
Small riprap - - Large uniform	3.9 ok	6 ok
Large graded - - Large uniform	.9 ok	3 ng

[1] No good.

are those given in the "Handbook of Applied Hydraulics," edited by Calvin V. Davis and Kenneth E. Sorensen, third edition, 1969, pages 18-25. Similar criteria are also given in "Design of Small Dams," U.S. Bureau of Reclamation, first edition, 1960, page 174. The filter material criteria are:

	Davis and Sorensen	USBR
$\frac{D_{15}\text{ filter}}{D_{85}\text{ base}}$	< 4 to 5	< 5
$\frac{D_{15}\text{ filter}}{D_{15}\text{ base}}$	> 4 to 5	= 5 to 40

The USBR criteria also state that the grain-size curves of the filter and the base be roughly parallel. This can be indicated by the standard deviation $\sigma = D_{15}/D_{50} = D_{50}/D_{85}$ because, for the grain size curves to be parallel for different materials, σ must be equal for the different materials.

INITIAL TESTS

After test 1, conducted with the initial crest shape, and after the approach velocity distribution had been improved, further formal testing was deferred in favor of exploratory tests to

determine the feasibility of "choking" the crest and to develop satisfactory stilling basin sidewall configurations.

Initial Structure Proportions

Test 1 was conducted using the structure proportions shown in figure 1. The streambed was coarse sand initially shaped as shown in figure 3. The design discharge was 11,200 c.f.s. and the duration 1.6 prototype days. Figure 5 shows the resulting scour.

The asymmetrical scour around the inlet shown in figure 5, A, occurred because of asymmetrical distribution of the velocities in the approach channel. The greatest depth of scour was to elevation 195.7, 8.3 feet below the crest. The dike eroded to the cutoff wall on both sides of the inlet and wave wash damaged the dike, particularly on the right side. The tops of the upstream channel banks eroded as a result of overbank flow returning to the channel. The possibility of this erosion had been anticipated by the State Design Unit.

The scour in the downstream channel is shown in figure 5, B. The greatest depth of scour was to elevation 184.6, 7.4 feet below the end sill and original bed elevation. The deepest scour occurred along the wingwalls. Since the bottoms of the wingwalls were at elevation 185, the left (looking downstream) wingwall was undermined and the right wingwall nearly so. There was little erosion of the dike. The local widening of the stream channel near the stilling basin exit had been anticipated. One object of the tests was to determine the proportions of this channel enlargement. Much of the scoured material was deposited in the downstream channel. The original channel bed was at elevation 192 and the final bed at about elevation 198.

Additional upstream and downstream scour would have occurred if the tests had been continued. However, the length of test 1 was sufficient to indicate that it would be desirable to choke the inlet to insure bank-full flow at the structure at bank-full channel discharge and that the downstream wingwall proportions needed revision. These revisions were made after the approach channel velocity distribution had been improved by adding baffles and screens as mentioned previously.

Compound Trapezoidal Weir

To decrease the structure capacity at bank-full stage (elevation 213) to the bank-full channel capacity of 4,000 c.f.s., the sides of the drop inlet were raised to elevation 213 as shown in figure 6. Since the computed capacity of the transverse weir was less than 4,000 c.f.s. at bank-full stage, the side weirs were cut back 6 feet at the top to give additional capacity.

The performance and the capacity of this weir were checked without repairing the scour that had occurred during test 1. The per-

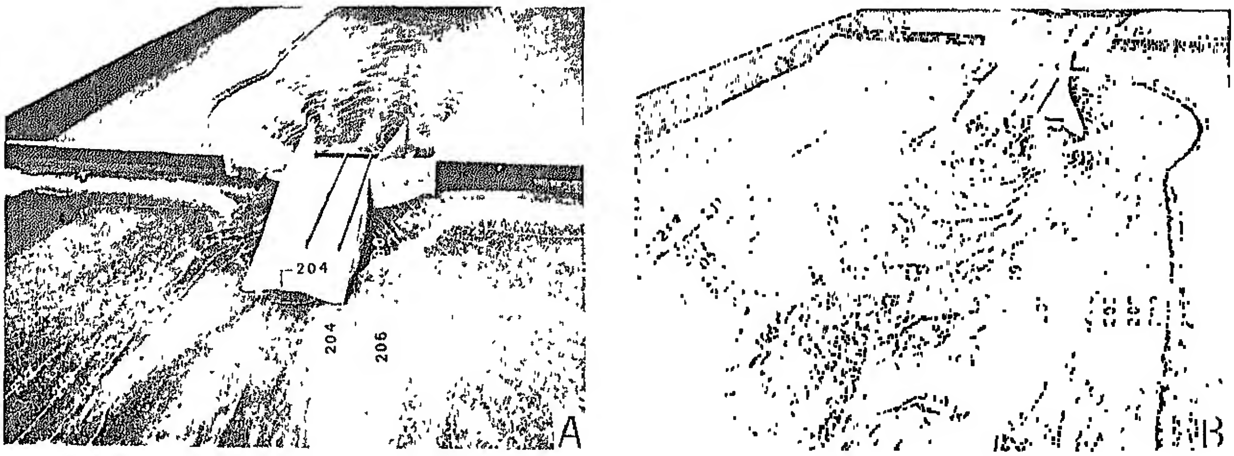


Figure 5.—Scour at initial structure (test 1: design discharge 11,200 c.f.s. for 1.6 days): A, Upstream channel; B, downstream channel.

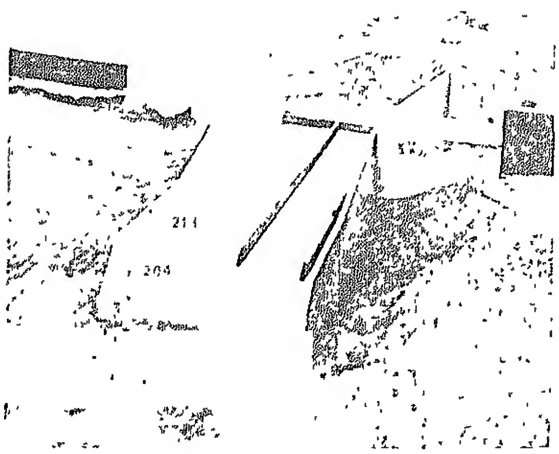


Figure 6.—Initial compound trapezoidal weir.

formance at the bank-full discharge of 4,000 c.f.s. is shown in figure 7, A. The headwater surface elevation is 213.4 feet, slightly greater than bank full. The performance at the design discharge of 11,200 c.f.s. is shown in figure 7, B. The headwater surface elevation of 219.0 feet is greater than desired, but it was accepted by the Mississippi SCS engineers because the choked weir completely eliminated dropdown of the overbank flow as it entered the channel near the structure.

All subsequent tests were made with the compound weir, although the cutback of the sides was changed from 6 to 7 feet to increase the structure capacity. The procedure for proportioning the choked trapezoidal crest and computing its capacity is given in "Head-Discharge Equations," page 42.

Stilling Basin Wingwalls

Figure 5, B, shows that the wingwalls were undermined after only 1.6 days at the design discharge. The deep scour at the end of the wingwalls also suggested that they were not long enough.

The first change was to decrease the wingwall top slope from 1 on 1 to 1 on 2. These wingwalls were undermined after 3.4 days at the design discharge as shown in figure 8, A. Prior to the test, sand had been placed to fill the space in back of the wingwalls; scour of this sand measured the protection provided by the wingwall. Most of this sand was washed out, probably passing under the wall. In back of the wingwall the eroded bank stood at the angle of repose as shown in figure 8, B. Because figure 8 shows that the end of the wingwall is almost buried in the bank, it appears that the top slope and length of the wingwall were probably about right.

The second change was to deepen the cutoff under the wingwalls and downstream end of the stilling basin. Sand was backfilled behind the wingwalls and the channel near the stilling basin exit was overfilled. After 2.9 days at the design discharge, scour had occurred along the face of the wingwalls to elevation 180.1 on the

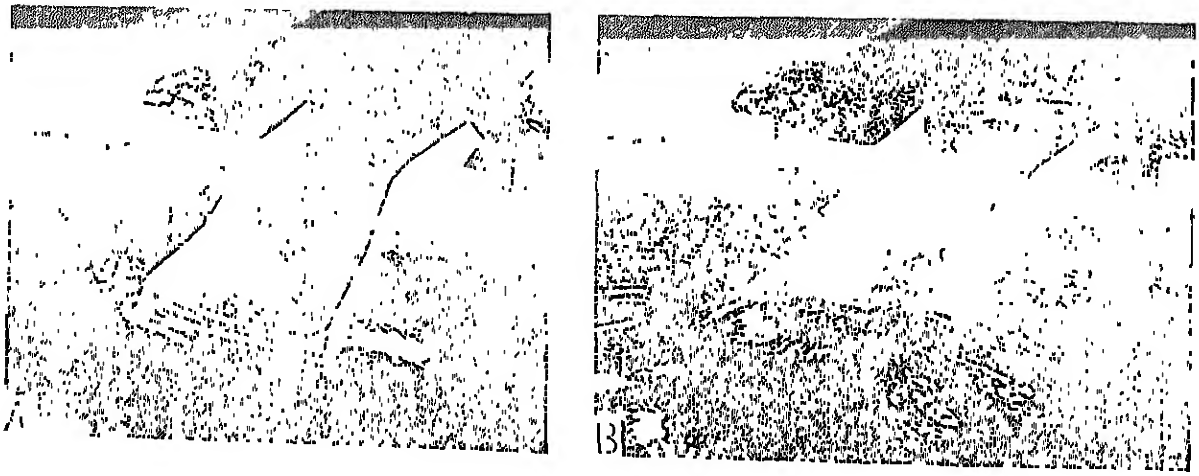


Figure 7.—Flow over initial compound trapezoidal weir: A, Bank-full discharge (4,000 c.f.s.; headwater surface elevation 213.4 ft.); B, design discharge (11,200 c.f.s.; headwater surface elevation 219.0 ft.).

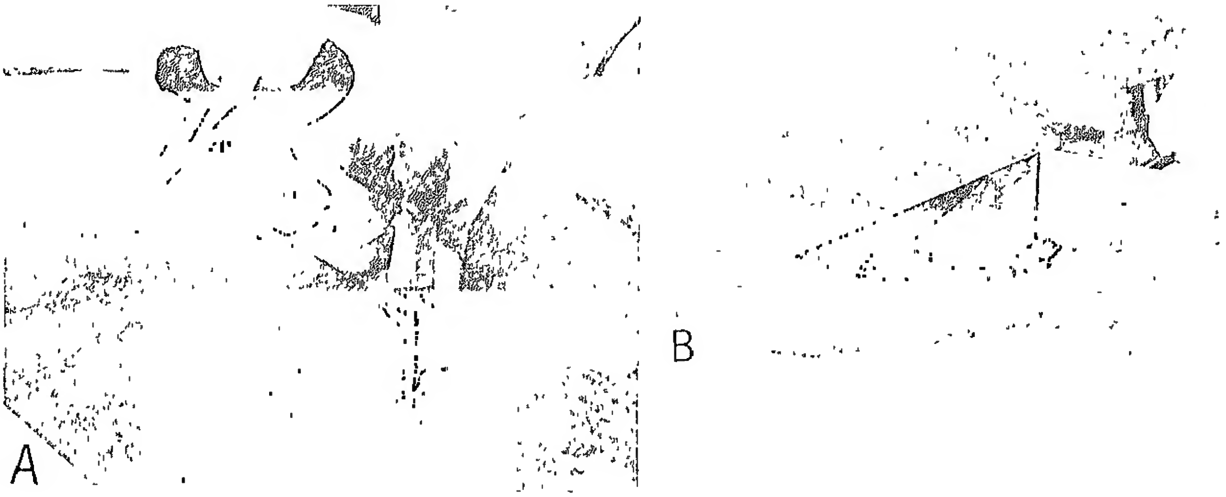


Figure 8.—Scour undermines 2:1 wingwalls after 3.4 days at design discharge: A, Sheet of paper is under right wingwall and fingers are under left wingwall; B, scour behind left wingwall.

right wall and elevation 178.4 on the left wall. The deepest scour occurred near the midlength of the wingwalls as shown in figure 9, A. Except opposite the deep scour along the face of the wingwall, the sand in back of the wingwall approximates the wingwall top slope as shown in figure 9, B. The scour pattern shown in figure 9 indicates again that the wingwall length and top slope are about correct.

A third change was to return to the 1-on-1 wingwall top slope, extend the wingwall until its top slope intersected the elevation of the stilling basin floor, and deepen the cutoff walls. This change was made to see if the shorter wingwall would be satisfactory if undermining were prevented. The scour after 2.9 days at the design flow is shown in figure 10, A. The greatest depth of scour along the wingwall was to elevation 183.0 on the right wall and to elevation 183.2 on the left wall. Figure 10, B, shows that the deep scour extended in back of the wingwall.

Increasing the time of scour to 11.4 days resulted in some additional scour of the bed and the dike as shown in figure 11. The maximum depth of scour along the right wingwall was to elevation 178.2, an increase of 4.8 feet, and along the left wingwall was to elevation 181.0, an increase of 2.2 feet for the 11.4-day period as compared with the 2.9-day period.

Figures 10 and 11 show that the wingwall with the 1-on-1 top slope permits more scour than can be tolerated. The wingwall with the 1-on-2 top slope shown in figure 9 was therefore used for all subsequent tests made to

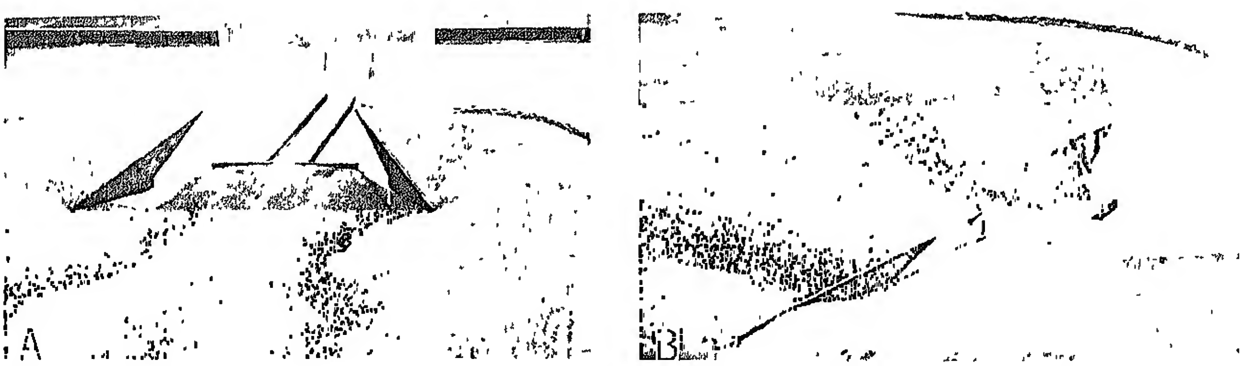


Figure 9—Scour near 2:1 wingwalls with deepended cutoff walls after 2.9 days at design discharge: A, Scour along face of wingwall; B, minimal scour behind left wingwall.

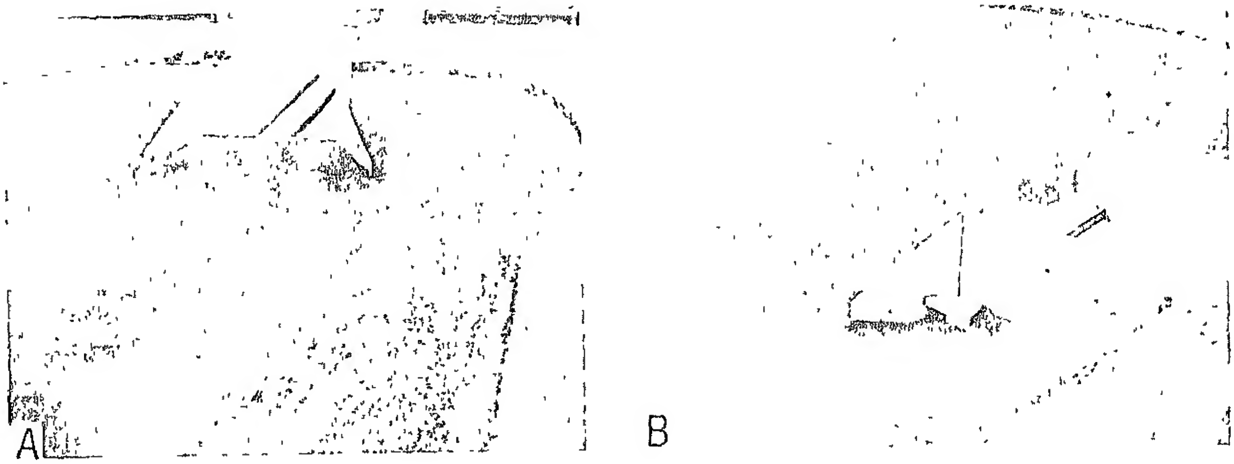


Figure 10.—Scour near 1:1 wingwalls with deepened cutoff walls after 2.9 days at design discharge: A, Appearance of dike and downstream channel; B, scour behind wingwall.

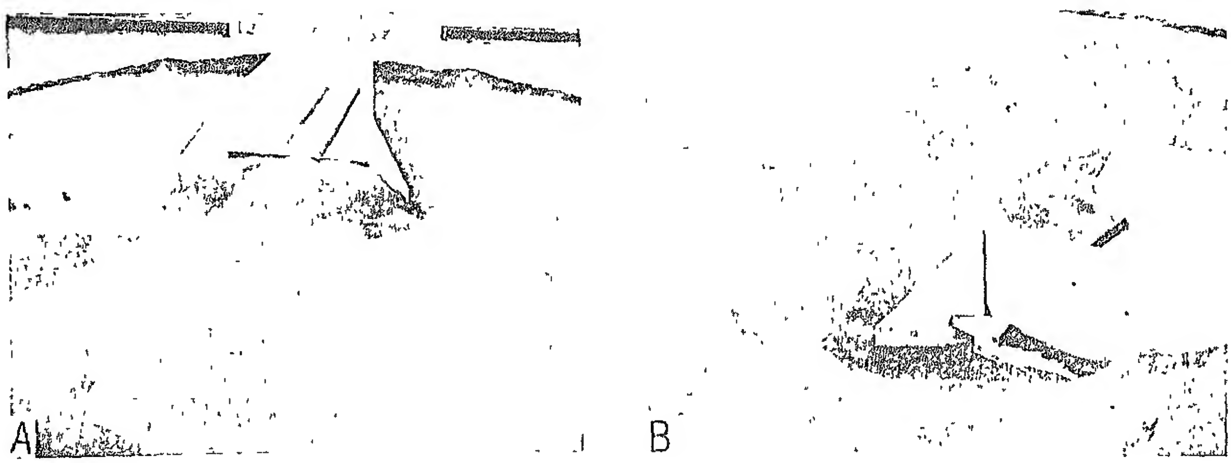


Figure 11.—Scour near 1:1 wingwalls with deepened cutoff walls after 11.4 days at design discharge: A, Appearance of dike and downstream channel; B, scour behind wingwall.

evaluate the downstream channel scour and scour protection.

Upon completion of the initial tests, attention was directed to the prevention of scour in the approach channel.

APPROACH CHANNEL TESTS

The choked trapezoidal crest with a 7-foot cutback of the high sidewalls was used during the tests to determine both the magnitude of the scour problem in the vicinity of the crest and the riprap size and placement required to solve the problem.

Scour Near Crest

The upstream channel was shaped in compacted coarse sand. Its appearance before the tests is shown in figure 12. For test 2 the design discharge (11,200 c.f.s.) scoured the bed

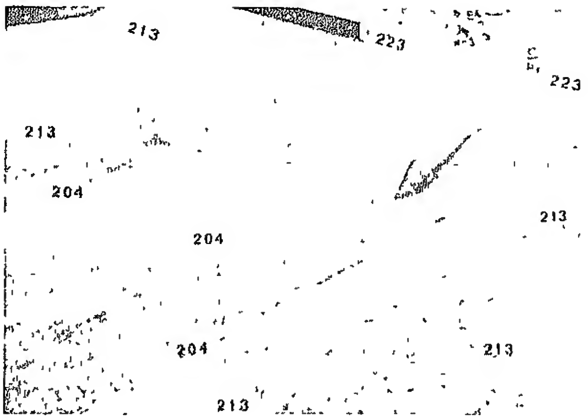


Figure 12.—Inlet and approach channel before tests.

for 2.9 days. After contouring, mapping, and photographing the scour, the design discharge was established for test 3. After a total scour time of 11.3 days, the flow was stopped and the bed again contoured, mapped, and photographed.

The scour after 2.9 days is shown in figure 13, A, and after 11.3 days in figure 13, B. Contour maps of the scour are shown in figure 14. It is obvious that serious scour can occur in the vicinity of the box inlet. After 2.9 days scour had occurred 40 feet upstream from the crest, and this distance increased to 50 feet at the end of 11.3 days. The width of the scour at the crest elevation after 2.9 days was about 87 feet and after 11.3 days about 105 feet. The greatest depth of scour was at the upstream corners of the box inlet and reached 10.9 feet below the spillway crest after 2.9 days and 14.0 feet after 11.3 days. The scour was rapid at first, but its rate decreased with time.

These tests with the design discharge and the channel shaped in coarse sand indicated a need for riprap protection for 50 feet upstream of the upstream weir crest, for a width of 105 feet at the crest elevation, and along the sides of the box inlet. It was later discovered that the protected area could be reduced because once the deep scour was prevented the sloughing that enlarged the scour hole was also prevented.

Determination of Riprap Size and Placement

To prevent scour, coarse graded riprap was placed over the 80-foot width of the approach channel bottom for 50 feet upstream and 8 feet downstream of the upstream crest of the box inlet. The channel was reshaped in coarse sand to its design dimensions. Operation at the design discharge for 3 prototype hours (test 4) produced no scour, although a few stones did pass through the spillway. It was obvious that the riprap was larger and covered a greater area than was necessary to prevent scour and protect the structure.

Using information and procedures developed by A. G. Anderson of the St. Anthony Falls Hydraulic Laboratory, K. Yalamanchili determined the incipient velocity of movement for each of the bed materials available at the laboratory. From isovels of the mean velocities measured over the bed used for test 4, the boundaries of the areas in which each bed material would not scour were determined. A riprap plan was developed from this information. (For an example, see p. 20.)

The riprap plan prior to test 5 is shown in figure 15. Coarse graded riprap was used 9 feet upstream of the crest and around a 9-foot radius from the ends of the crest. Small riprap was placed outside the coarse graded riprap and inside a 36-foot radius arc from the center of the crest. Small riprap was carried up the dike within a line that intersected the crest at the elevation of the water surface for the design discharge (elevation 219.0) and was tangent to the arc. The channel was shaped in coarse sand.

The appearance of the approach channel and riprap after 2.9 days at the design flow (11,200 c.f.s.) is shown in figure 16. The fines have been removed from the coarse sand just upstream of the riprap. The maximum depth of scour is 3 inches (to elevation 203.76). In the vicinity of the crest the small stones in the coarse graded riprap have been eroded. The maximum scour is 1.6 feet (to elevation 202.4) at the left end of the crest. Because of the rapid acceleration of the flow near the crest and the relatively large current meter, it was not possible to measure the local velocities there. This, of course, prevented determining the riprap size required. Also, near the intersection of the crest and the water surface the shallow depth prevented velocity measurements and determining the required riprap size. Scour there shows that the riprap used was too small.

The riprap shown in figure 16 on the paper

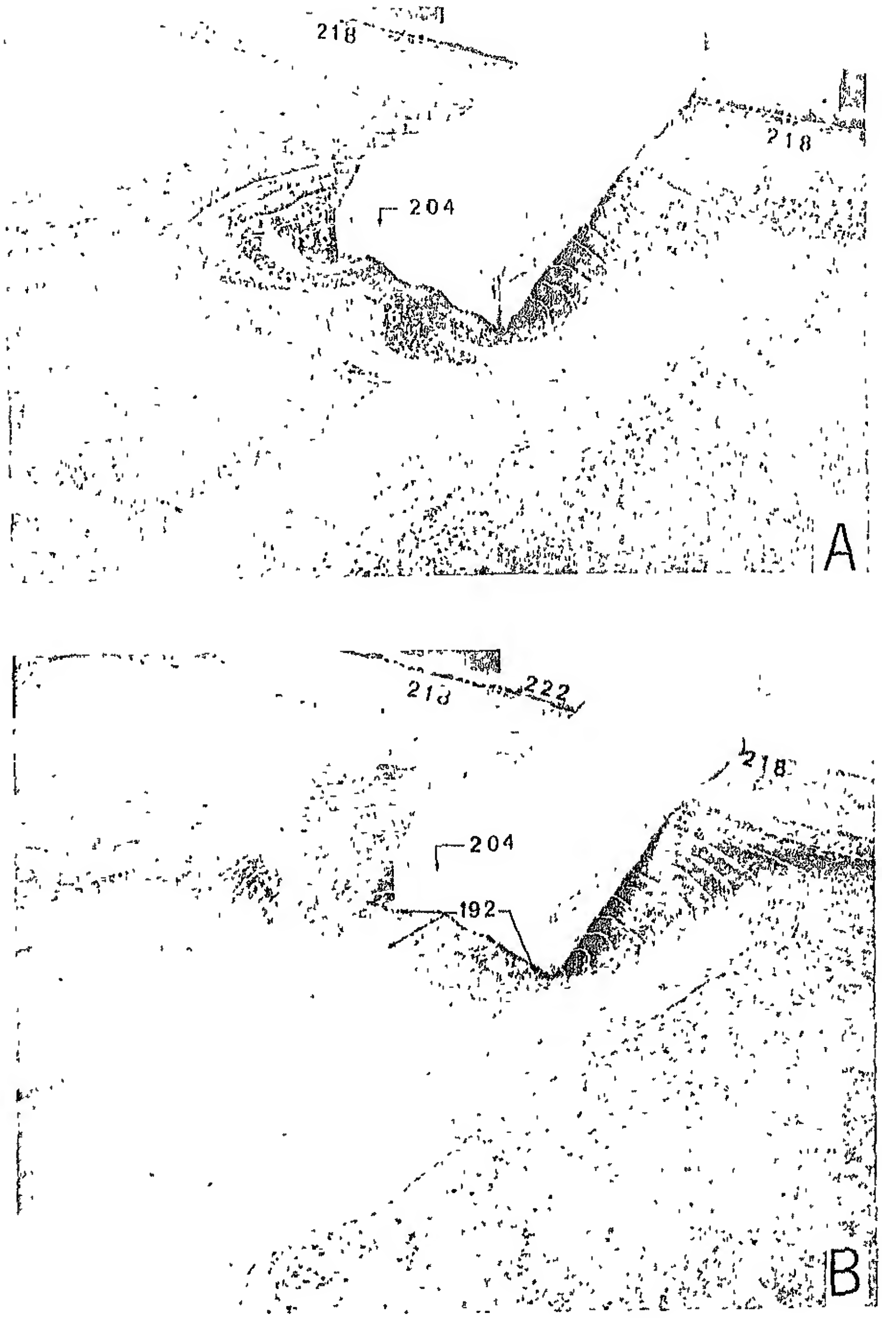


Figure 13.—Scour in coarse sand near box inlet caused by design discharge: A, After 2.9 days (test 2); B, after 11.3 days (test 3).

placed in the box inlet was removed from the stilling basin. It consists of sand probably eroded from beside the box inlet near the water surface and from the approach channel, some small riprap eroded mostly from near the water surface beside the box inlet, and some of the smaller coarse graded riprap stone eroded from close to the upstream crest.

In general, the riprap arrangement used for test 5 was considered satisfactory. However, for test 6 the riprap design was revised to place the largest stones available in the coarse graded riprap for a distance of 3 feet upstream from the upstream creast. This riprap size was about 27 inches (984 lb.) and was carefully hand placed with its long axis vertical to develop its

maximum resistance to uplift and scour. In addition, the small riprap along the sides of the box inlet was replaced with coarse graded riprap from elevation 212 to elevation 219. The remainder of the riprap was left in the condition following test 5. However, the coarse sand channel was reshaped before test 6 as is shown in figure 17.

The design flow (actually 11,360 c.f.s.) is shown in figure 18. Confetti shows the surface currents in this 1/6-second (prototype time) exposure. The contoured upstream channel after 11.2 days is shown in figure 19. There has been some surface movement of coarse sand, particularly on the dike near the crest, and some wave wash on the dike. However, it should be kept in mind that the sizes of riprap used include no safety factor; the riprap was sized for imminent movement and the sizes shown are probably barely adequate. A safety factor should be applied to the sizes shown when the prototype riprap protection is designed.

The riprap protection to the upstream channel shown in figures 17 and 19 is considered satisfactory. It was left in place for the ensuing tests and demonstrations, which covered a 3-month period. The riprap provided adequate protection and did not deteriorate during that period.

The large selected hand-placed riprap close to the upstream end of the crest and along the sides of the box inlet can be scaled up from the model size to give the prototype sizes for *imminent* movement. Individual pieces of this riprap weigh about 1,000 pounds and cover the

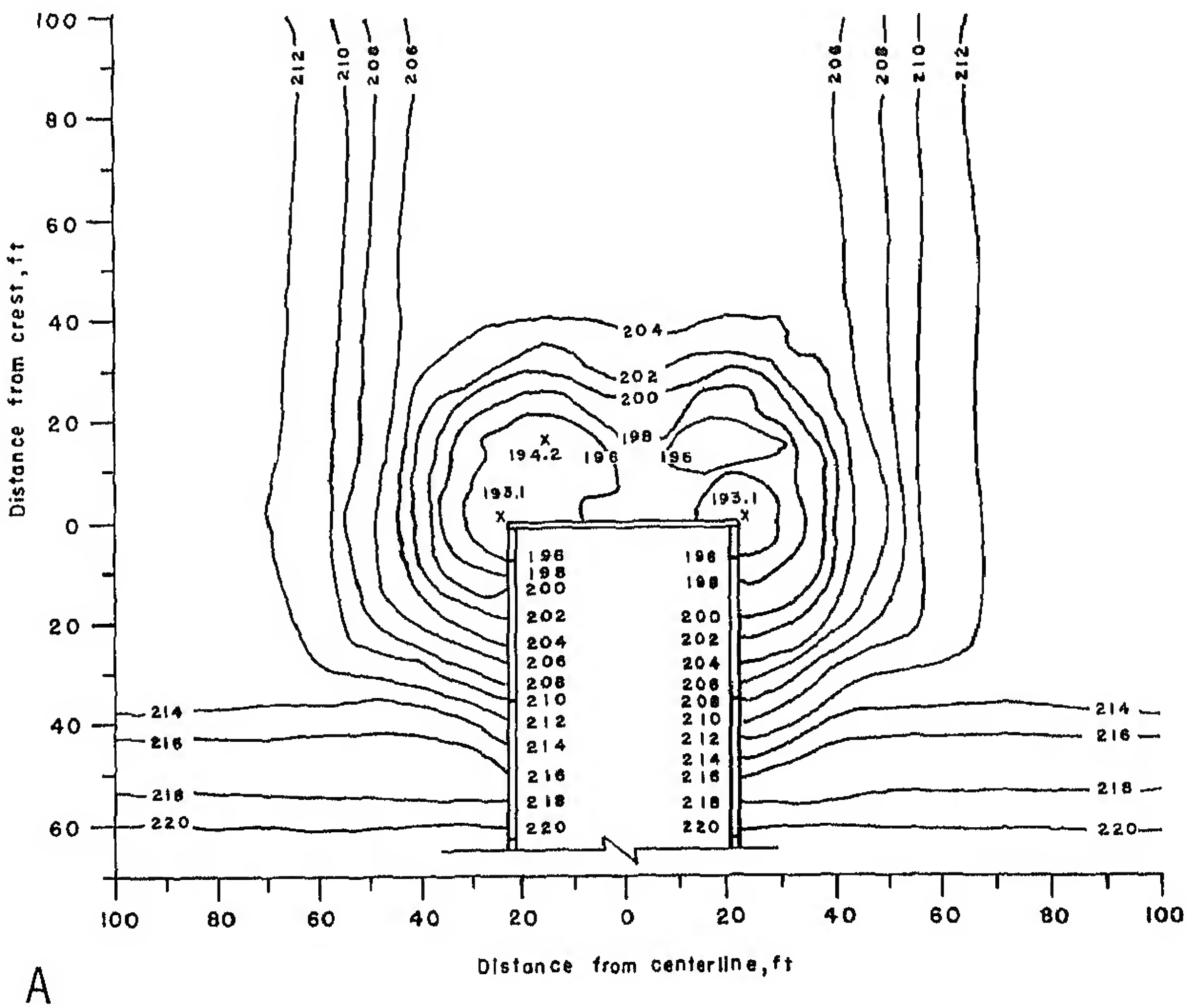


Figure 14, A.—Scour near box inlet caused by design discharge, after 2.9 days (test 2).

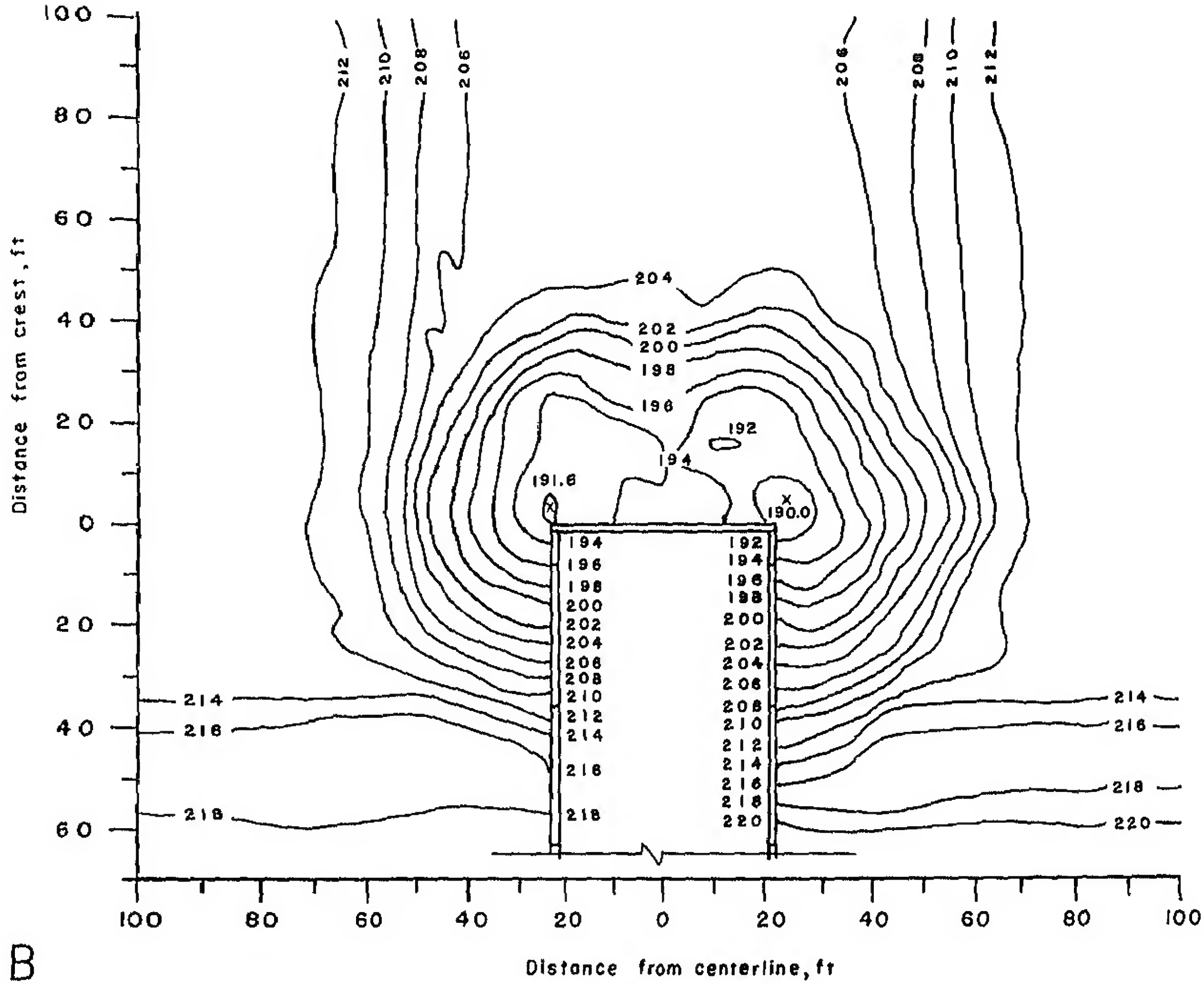


Figure 14, B.—Scour near box inlet caused by design discharge, after 11.3 days (test 3).

bed for a distance of 3 feet from the crest. Increased sizes should be used to assure no riprap movement. The other riprap sizes and the other areas to be protected should be designed using procedures given in "Design of Riprap," page 22. The design procedure requires a knowledge of the velocities and the velocity distribution in the approach channel. The required information follows.

Approach Channel Velocity Distribution

The bed and banks in the approach channel may be stable for the design discharge and velocities. However, because the flow accelerates as it approaches the box inlet, the resulting higher velocities in the approach may scour the channel bed in the vicinity of the drop inlet. This has been amply demonstrated in "Scour Near Crest," page 12. In order to design the riprap required to protect the bed from scour, it is necessary to know the actual velocities in the approach to the box inlet.

The velocities in the approach to the box inlet were measured during test 10 for the design discharge (11,200 c.f.s.) and during test 11 for the bank-full discharge (4,000 c.f.s.). These velocities (V = the velocity measured at the 0.6 depth or the average of the velocities measured at the 0.2 and 0.8 depths) were divided by the average velocity in the approach channel ($\overline{V}$ = the discharge divided by the flow cross section area) to obtain a multiplication factor that can be applied to the average

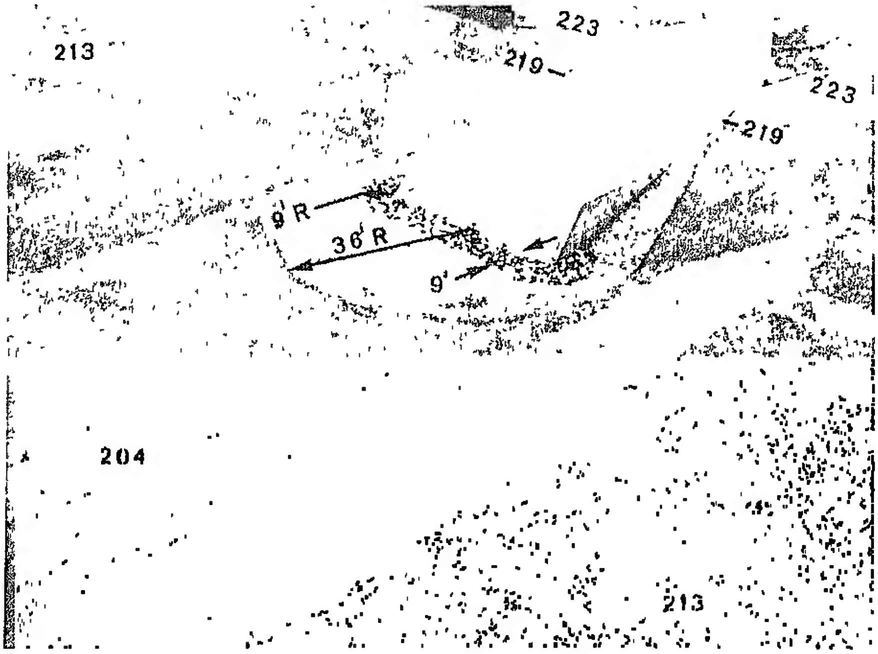


Figure 15.—Initial upstream riprap placement before test 5.

prototype approach channel velocity to obtain the actual velocity in the prototype. Multiplication factor isovels are given in figure 20 for the design discharge and in figure 21 for the bank-full discharge.

The isovels could have been plotted for the

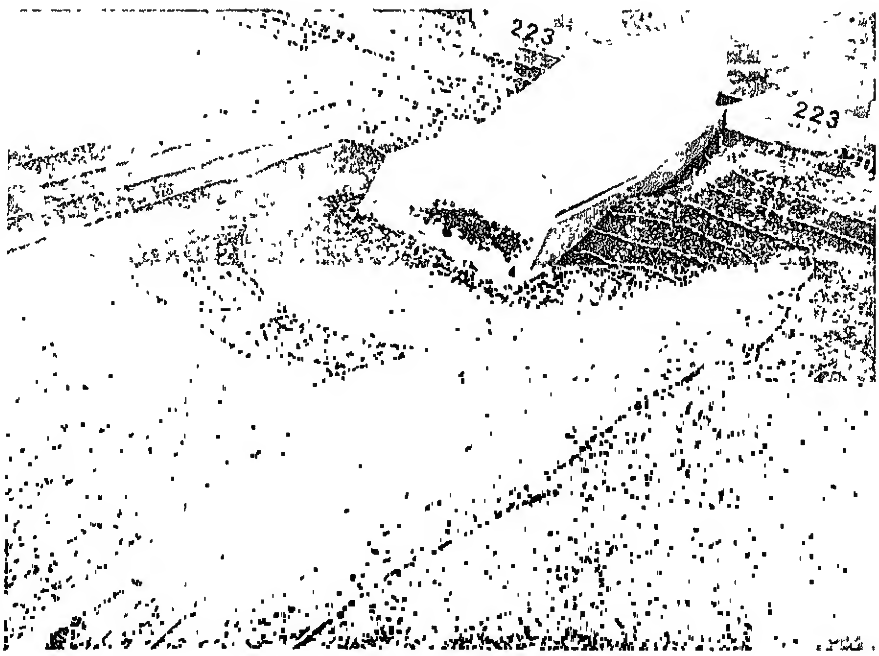


Figure 16.—Appearance of upstream channel after test 5 (design discharge 11,200 c.f.s. for 2.9 days). Some riprap has scoured from near crest and near water surface.

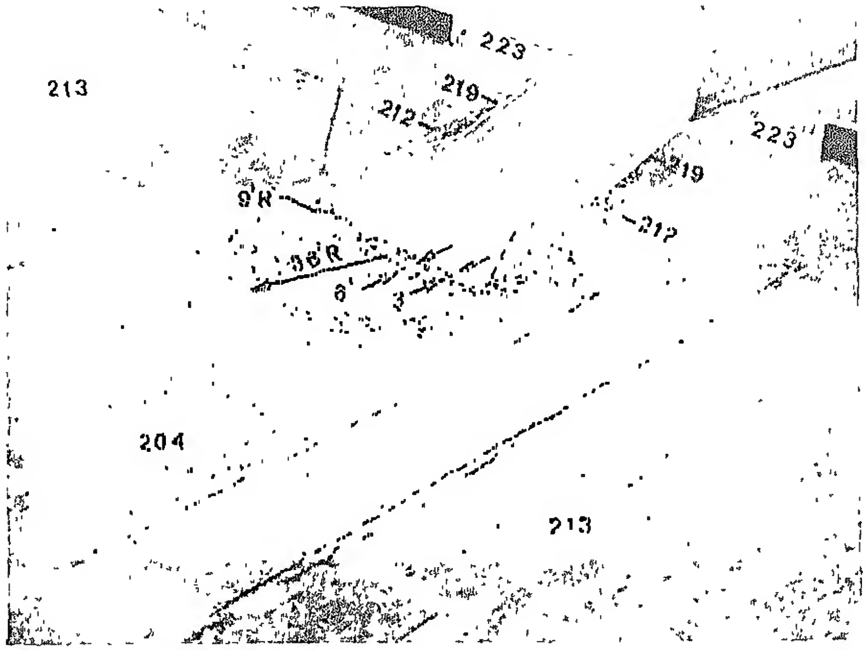


Figure 17.—Final upstream riprap placement before test 6. Channel is formed in coarse sand. Small riprap is placed inside a boundary formed by a 36-foot radius arc from center of upstream crest and a tangent to the arc extending to design pool level at elevation 219.0. Downstream boundary of small riprap and upstream boundary of coarse graded riprap is a line 9 feet upstream of upstream crest, which extends on a 9-foot radius around ends of crest and along sides of box inlet to elevation 212.0. Coarse graded riprap is also used along sides from elevation 212.0 to elevation 219.0. Selected large stone is placed with its major axis vertical for a distance of 3 feet from upstream crest and for a radius of 3 feet from ends of crest.

prototype velocities. However, the dimensionless procedure was adopted to make the results more generally applicable—so they could be used to estimate the actual velocities at other locations where the average velocity in the approach might be different from that at the structure modeled. To obtain actual velocities for the compound trapezoidal box inlet drop spillway modeled, the multiplication factor isovels in figure 20 for the design discharge

Figure 18.—Design flow into compound trapezoidal weir box inlet drop spillway during test 6 (discharge 11,360 c.f.s.; headwater elevation 219.0 ft.; exposure 1/6 sec.). Confetti shows surface currents.

Figure 19.—Appearance of upstream channel after test 6 (design discharge—actually 11,360 c.f.s.—for 11.2 days). This shows satisfactory performance of final upstream riprap placement.

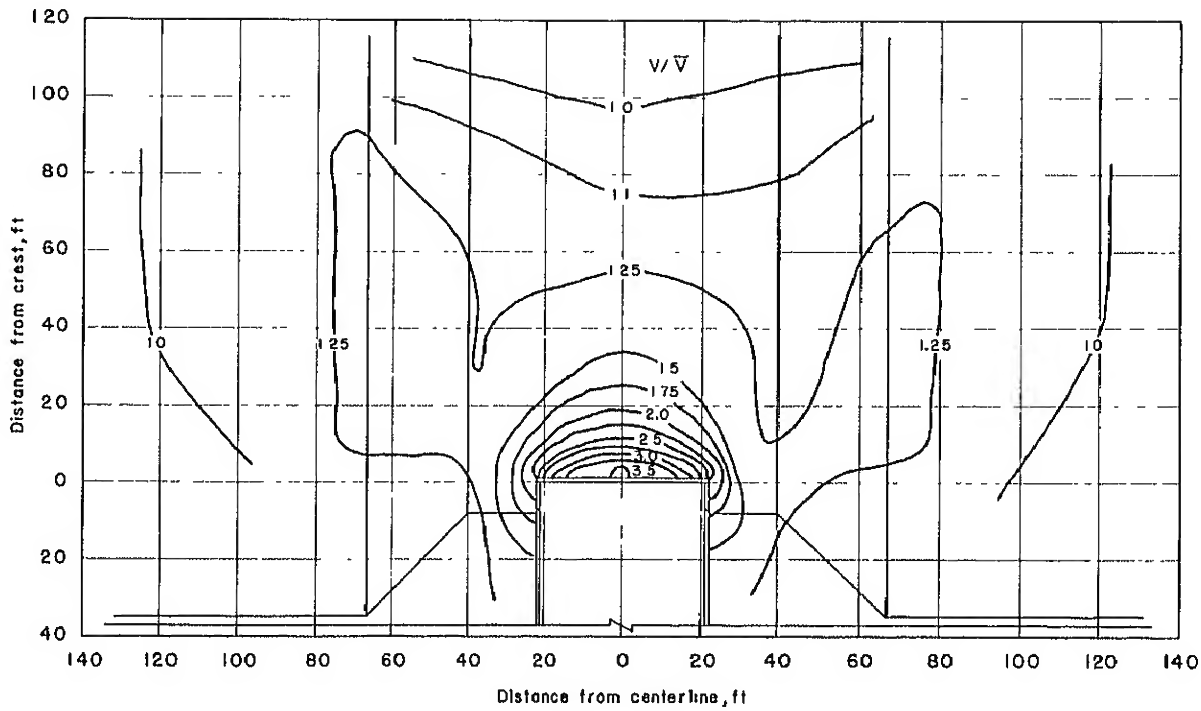


Figure 20.—Relative velocity distribution in approach channel in terms of average velocity for design discharge.

should be multiplied by 3.85 prototype f.p.s. For figure 21 the bank-full prototype average velocity was 4.16 f.p.s. These multipliers were scaled up from the model. The actual average design discharge velocity in the prototype may differ somewhat from the model velocity because of differences in the overbank flow area.

Figures 20 and 21 are for use in determining

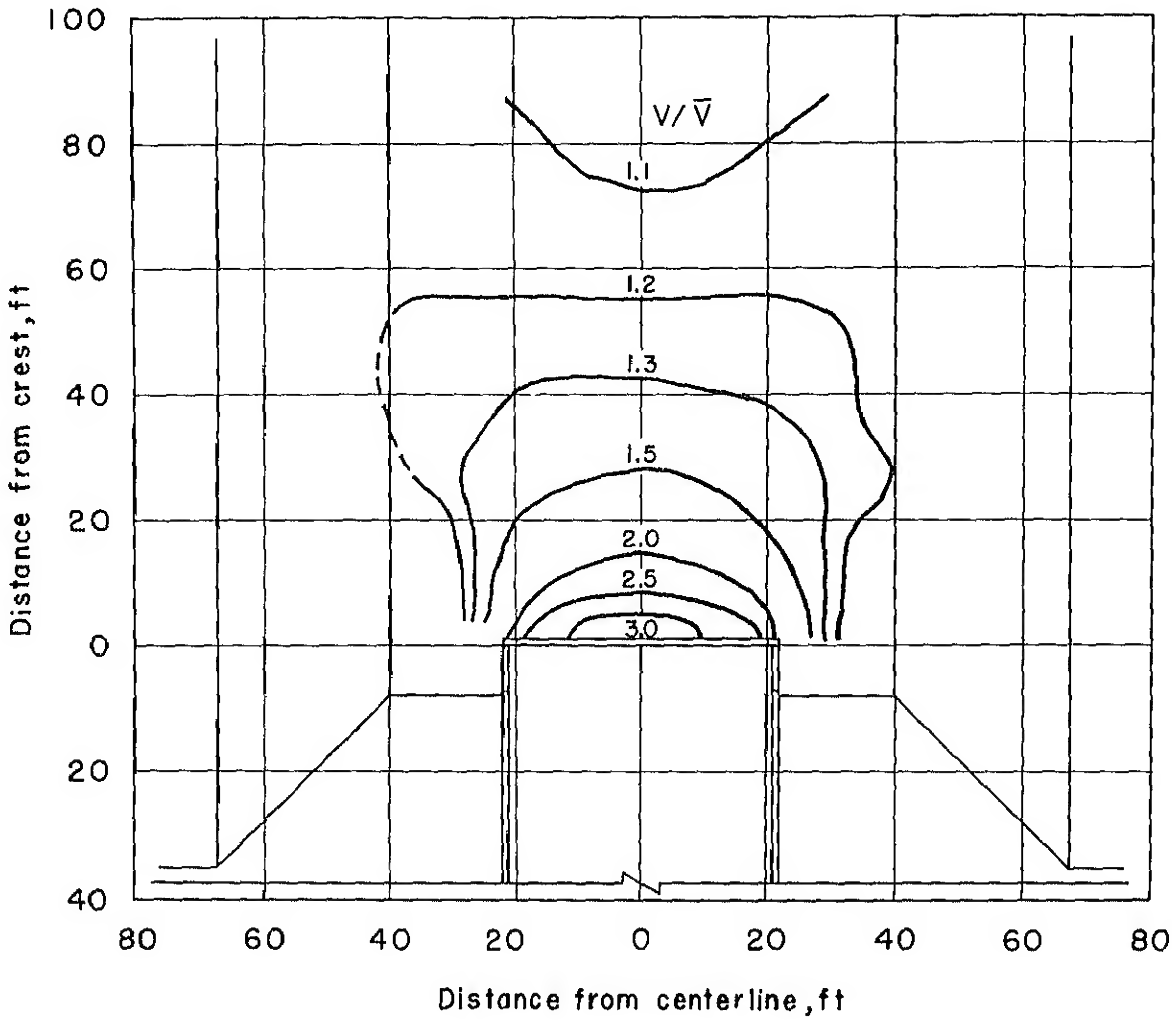


Figure 21.—Relative velocity distribution in approach channel in terms of average velocity for bank-full discharge.

the riprap placement in the channel approaching the box inlet drop spillway. The procedure used, if applied to the prototype, is:

(1) For a selected size of riprap, determine the velocity for imminent movement of the riprap.[2] This determination is made using the method presented in "Design of Riprap," page 22.

(2) Multiply this velocity by the safety factor to obtain the prototype design velocity V.

(3) Divide the design velocity by the average velocity in the approach channel $\overline{V}$ to obtain a value of $V/\overline{V}$.

[2] An alternative procedure would be to select the riprap size required for a given velocity. Use of this alternative procedure would necessitate modification of the procedure used here.

(4) On figures 20 or 21 draw the relative isovel corresponding to $V/\overline{V}$. This isovel will define the downstream boundary of the selected riprap. Higher velocities downstream of this boundary will require the use of heavier riprap.

This procedure was followed to design the riprap used in the model except no safety factor was applied. As noted previously, the model riprap design was satisfactory.

Example.—The design of the model riprap placement will be used to illustrate the procedure.

The isovels used for the model riprap placement design are shown in figure 22. (This was an early measurement. More detailed measurements were made for figs. 20 and 21.)

The velocities for imminent movement of

the available riprap were determined by the method presented in "Design of Riprap," page 22. For the 0.82-foot depth of flow measured in the model they are:

Bed material	Size (mm.)	Velocity (f.p.s.)
Coarse sand	0.70	1.38
Small riprap	4.25	2.53
Large graded riprap	12.7	3.64
Large uniform riprap	22.5	4.40

The velocity for imminent movement of the coarse sand is 1.38 f.p.s. An examination of the isovels in figure 22 shows that an arc of 2.0-foot (36-ft. prototype) radius from the center of the box inlet crest will approximate the 1.38-f.p.s. isovel, so small riprap was placed inside this arc. Rather arbitrarily the small riprap was also placed inside a line originating at the intersection of the water surface and the box inlet sidewall and tangent to the arc. (Later tests showed heavier riprap was needed near the water surface, but the shallow depths

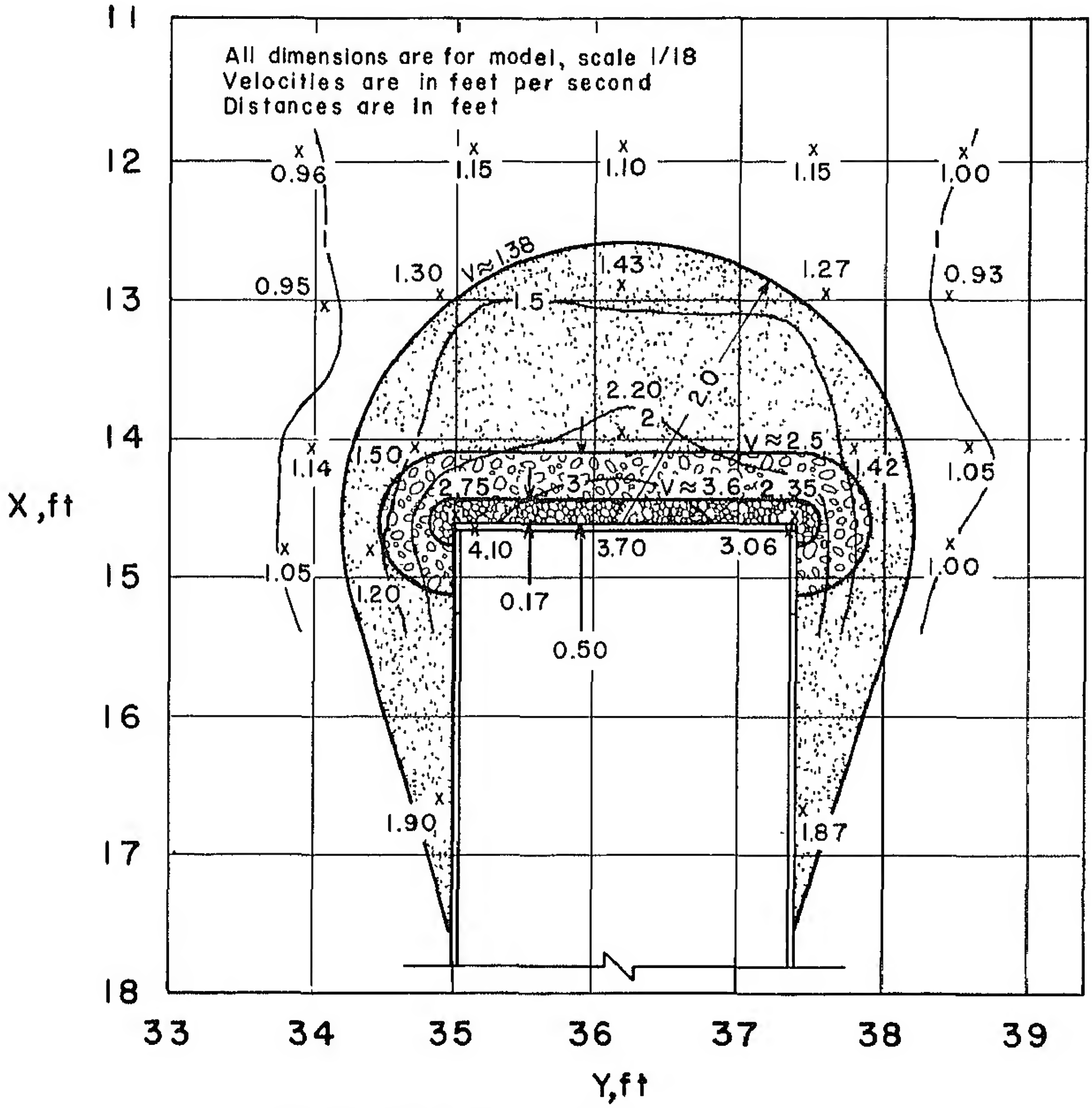


Figure 22.—Isovels and riprap placement for the model.

there prevented current meter measurements of the velocities necessary for the riprap design.)

The velocity for imminent movement of the small riprap is 2.53 f.p.s. A second examination of the isovels showed that this velocity occurs at the channel centerline about 0.5 foot (9-ft. prototype) upstream from the crest. The boundary between the small riprap and the large graded riprap was therefore set at 0.5 foot upstream of the crest and the boundary was extended as an arc of 0.5-foot radius around the ends of the crest.

Similarly the velocity for imminent movement of the large graded riprap is 3.64 f.p.s. After again examining the isovels, the downstream limit of the large graded riprap was set at 0.17 foot (3-ft. prototype) from the crest.

Finally large uniform riprap was hand placed for a distance of 0.17 foot (3-ft. prototype) upstream from the crest because its velocity for imminent movement of 4.40 f.p.s. exceeded any measured velocity.

Since only minor movement of this riprap occurred during subsequent tests and demonstrations,[3] it can be concluded that the design procedure which determined the placement of the several sizes of riprap is satisfactory.

It will be mentioned again that this satisfactory riprap design was based on velocities such that the riprap was in imminent danger of movement. Larger riprap employing an adequate safety factor should be used in the prototype.

Design of Riprap

Determining the size of riprap that will protect a streambed from a given velocity is based on a method described in a letter from A. G. Anderson, St. Anthony Falls Hydraulic Laboratory, to Melvin M. Culp, SCS, on December 8, 1965. The constants were, however, developed from a more recent report by Anderson et al.[4]

[3] See "Determination of Riprap Size and Coverage," p. 13, for the results of the tests.

[4] Anderson, A. G., Paintal, A. S., and Davenport, J. T. Tentative design procedure for riprap-lined channels. Natl. Acad. Engin. Natl. Coop. Highway Res. Program Rpt. 108, 75 pp. 1970.

The method is based on the Manning equation

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad (1)$$

where V is the average velocity in feet per second, n is the Manning roughness coefficient, R is the hydraulic radius in feet, and S is the slope of the energy grade line in feet per foot.

The Manning roughness coefficient n for riprap is, after Anderson et al. (see p. 6, equation 15, or p. 8, fig. 6, of ftnt. 4),

$$n = 0.0395\, D_{50}^{1/6} \qquad (2)$$

where D_{50} is the riprap size in feet of which 50 percent is finer by weight.

The hydraulic radius R is, in accordance with the custom for wide channels, taken equal to the depth of flow d, so

$$R = d \qquad (3)$$

The slope S is evaluated in terms of the riprap size and depth of flow in the following analysis.

The critical shear stress acting on a streambed is

$$\tau_c = \gamma d S \qquad (4)$$

where τ_c is the critical shear stress in pounds per square foot and γ is the specific weight of water in pounds per cubic foot. Rearranging this equation to solve for S gives

$$S = \frac{\tau_c}{\gamma d} \qquad (5)$$

After Shields,[5] Anderson et al. (see p. 9 of ftnt. 4) show that equating the drag force acting on a particle at incipient motion

$$C_D D_{50}^2\, \tau_c \qquad (6)$$

to the submerged weight of the particle

[5] Shields, A. Anwendung der Aehnlichkeitsmechanik und der Turbulenzforschung auf die Geschiebebewegung. Mitt. der Preuss. Versuchsanst. f. Wasserbau u. Schiffbau, pt. 26, 75 pp. Berlin. 1936.

$$C_W(\gamma_s - \gamma)D_{50}^3 \quad (7)$$

and subsequently rearranging the equality results in a ratio that they say can be taken as a constant for larger particles.[6] Thus

$$\frac{\tau_c}{(\gamma_s - \gamma)D_{50}} = \frac{C_W}{C_D} = C \quad (8)$$

from which

$$\tau_c = C(\gamma_s - \gamma)D_{50} \quad (9)$$

In these equations C_D incorporates the drag coefficient, shape factor, and fluid properties; C_W is a volumetric shape factor; $C = C_W/C_D$ is a constant for larger particles ($D_{50} > 0.001$ ft. = 0.3 mm.); and γ_s is the specific weight of the riprap in pounds per cubic foot.

Anderson et al. (see p. 10, fig. 8, of ftnt. 4) show that the critical shear stress as observed by 10 investigators using riprap ranging from 0.001 to nearly 0.5 foot can be represented by the equation $\tau_c = 5D_{50}$ (p. 9, equation 21, of ftnt. 4). However, they define τ_c "... as that value of the bed shear at which there is a general movement of the particles ..." (p. 9) and suggest that the constant should be reduced "... to some lower value to represent stone sizes that are stable ..." (p. 9). Their equation, which encompasses "... practically all the plotted data and should represent the upper limit of stable bed riprap" (pp. 9-10), is

$$\tau_c = 4D_{50} \quad (10)$$

(see p. 10, equation 22, of ftnt. 4).

Working through the equations to evaluate S and C, we obtain

$$S = \frac{\tau_c}{\gamma d} = \frac{4D_{50}}{\gamma d} = \frac{C(\gamma_s - \gamma)D_{50}}{\gamma d} \quad (11)$$

and, if γ_s = 165 pounds per cubic foot and γ = 62.4 pounds per cubic foot, which is assumed to apply to the bed material and water used for the test results summarized in the report by Anderson et al.,

$$C = \frac{4}{165 - 62.4} = 0.039 \quad (12)$$

When n, R, and S are substituted in the Manning equation (1)

$$V = \frac{1.486}{0.0395\,D_{50}^{1/6}}\,d^{2/3}\left[\frac{0.039(\gamma_s - \gamma)D_{50}}{\gamma d}\right]^{1/2} \quad (13)$$

which reduces to

$$V = 7.43\left(\frac{\gamma_s - \gamma}{\gamma}\right)^{1/2}\left(\frac{d}{D_{50}}\right)^{1/6}\sqrt{D_{50}} \quad (14)$$

or, for γ_s = 165 pounds per cubic foot and γ = 62.4 pounds per cubic foot,

$$V = 9.53\left(\frac{d}{D_{50}}\right)^{1/6}\sqrt{D_{50}} \quad (15)$$

when these equations are solved for the size of riprap,

$$D_{50} = 0.00244\left(\frac{\gamma}{\gamma_s - \gamma}\right)^{3/2}\frac{V^3}{\sqrt{d}} \quad (16)$$

and

$$D_{50} = 0.00116\,\frac{V^3}{\sqrt{d}} \quad (17)$$

It is suggested that one of the last four equations be used for designing riprap to protect the bed in the vicinity of box inlets. It will be necessary to use the equations containing γ_s only if the specific weight of the riprap is different from 165 pounds per cubic foot. It should be especially noted that since the equations give riprap sizes bordering on imminent movement, an adequate safety factor should be applied to the computed sizes.

Summary, Conclusions, and Recommendations

Excessive scour can occur in the vicinity of the inlet to a compound trapezoidal box inlet drop spillway. This scour can be prevented by

[6] Anderson et al. (see ftnt. 4) state on p. 9 and show in fig. 8, p. 10, that τ_c/D_{50} is a constant for values of D_{50} exceeding 0.001 ft. (0.3 mm.).

protecting the erodible approach bed with riprap of adequate size.

For the box inlet drop spillway modeled, it is recommended that selected riprap of 27 inches (984 lb.) minimum size, which should be increased to provide an adequate factor of safety, be placed for a distance of 3 feet upstream from the box inlet crest. The major axis of this riprap should be vertical.

It is further recommended that the riprap placement and size farther upstream than 3 feet from the box inlet crest be designed following the procedures in "Approach Channel Velocity Distribution," page 16, and "Design of Riprap," page 22.

DOWNSTREAM CHANNEL TESTS

No attention was given the downstream channel during the tests to experimentally design the approach channel protection. The channel was allowed to deteriorate. However, for test 6, the last test made to check the approach channel riprap plan, it was anticipated that no approach channel erosion would occur and that no further changes would be required in the approach channel; so test 6 was also made the initial test for the experimental design of the downstream channel.

Test 6 was conducted to determine the scoured shape of the downstream channel. From the scoured shape a riprap plan was devised to protect the downstream channel. This plan was checked in test 7. The riprap plan was then revised to correct weaknesses discovered during test 7 and to meet additional field requirements. The revised riprap plan—the plan eventually recommended—was checked in test 8. Each of these steps will be described in the following sections.

Scour of Sand Channel

The downstream channel for test 6 was formed in coarse sand. It was shaped as shown in figures 1 and 23. However, a change in the stilling basin was made to flatten the wingwall top slope from 1 on 1 as shown in figure 1 to a top slope of 1 on 2 as shown in figure 23. This

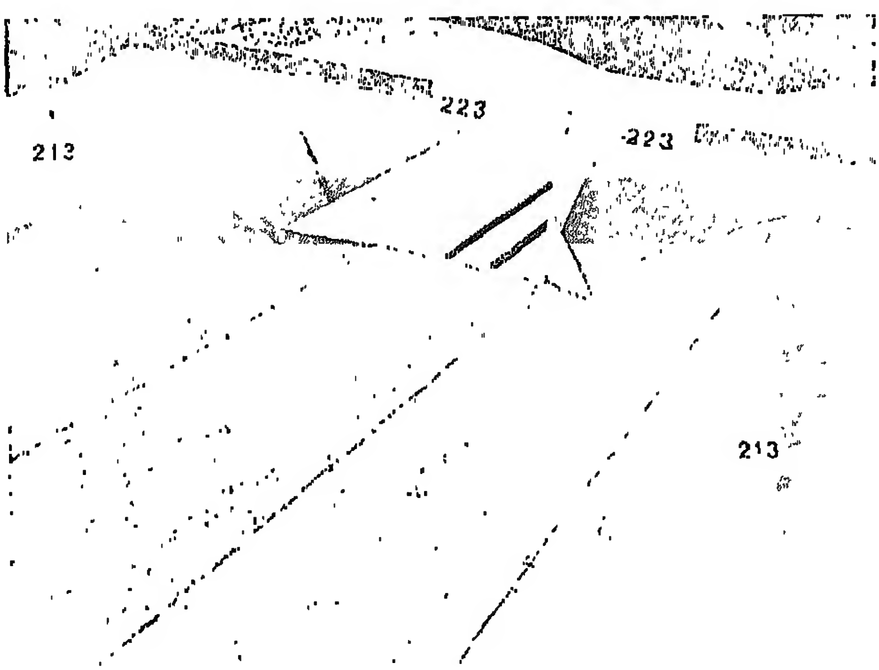


Figure 23.—Downstream channel shaped in coarse sand prior to test 6.

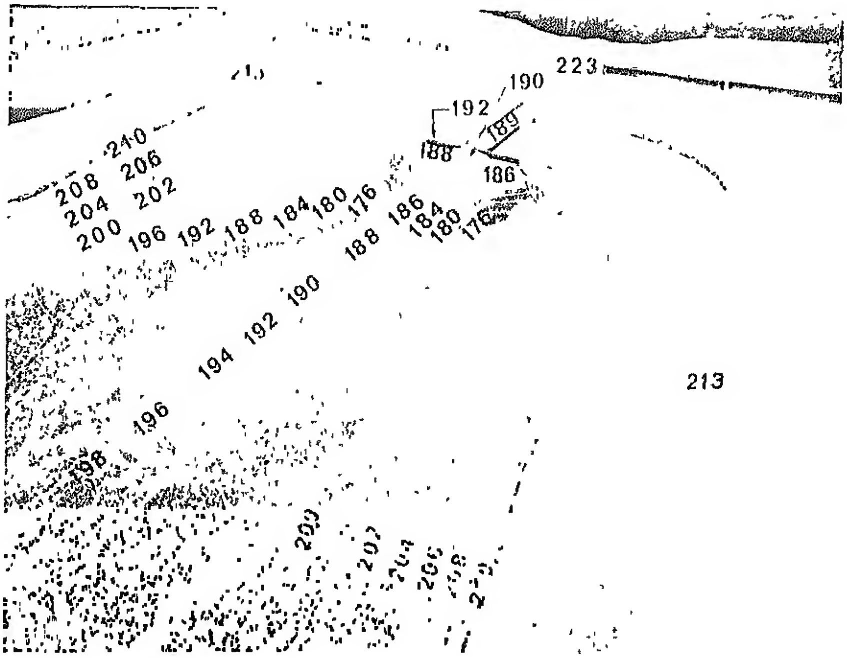


Figure 24.—Appearance of coarse sand downstream channel scoured by design discharge for 11.2 days (test 6).

change was made in accordance with the findings reported in "Stilling Basin Wingwalls," page 10.

The design discharge (actually 11,360 c.f.s.) was allowed to scour the downstream channel for 11.2 prototype days. Figure 24 shows the scoured bed. The light-colored material is fine sand. A deposit of fine sand can be seen on the downstream face of the dam near the waterline. This fine material was separated from the coarse sand and transported by water currents that moved upstream along the sides of the widened channel. There is no scour of the dam itself, but the downstream channel widened and deepened. Apparently the fine sand deposits on the downstream channel banks and bed occurred during the later stages of the scour. The changing velocity pattern as the scour progressed undoubtedly permitted the fine sand to be deposited where previously the channel had scoured.

A contour map of the scoured bed is shown in figure 25. As shown also in figure 24, the deepest scour occurred in line with the stilling basin sidewalls and opposite the ends of the wingwalls. The bed scoured to elevation 174.8, 17.2 feet below the original bed and the end sill elevations (elevation 192.0). At the center of the channel the maximum scour is to elevation 185.5, 6.5 feet below the end sill elevation. Little scour occurred close to the end sill. Along the faces of the wingwalls the scour depth increased to about the wingwall midlength and then decreased. There was no scour of the downstream dam face. In fact, mud and fine sand were deposited on the original surface of the dam.

Downstream of the stilling basin the bed scoured for approximately 90 feet. Scoured material was deposited on the original bed at greater downstream distances and reached a deposit depth of about 7 feet (to elevation 199 ±) about 280 feet downstream from the end sill.

Scour widened the channel at the stilling basin exit. This was expected, and determining the magnitude of the widening was one objective of test 6. The original channel width at elevation 213 was 186 feet and the greatest scoured width was 286 feet, an increase in width of 100 feet.

Scour also increased the width of the downstream channel throughout the length of the model, which extended 354 feet downstream of the end sill. The channel width near the end of the model was about 230 feet, an increase of about 44 feet.

The contour map of the scoured downstream channel shown in figure 25 was obtained to use as a basis for developing the riprap plan for the downstream channel. The development of this plan is explained in the following section.

Initial Riprap Placement Design

The planned test program for the downstream channel was based on the assumption that energy dissipation would be initiated but would not be completed within the stilling basin. Beyond the stilling basin the energy dissipation would be completed in an enlarged section of the downstream channel. The shape of this channel enlargement would be determined by letting the flow erode a sand-bed channel to form the "ideal" shape of the enlargement. This ideal shape would then be approximated by simple geometric forms. The geometrically described shape of the channel enlargement would then be used to transfer the

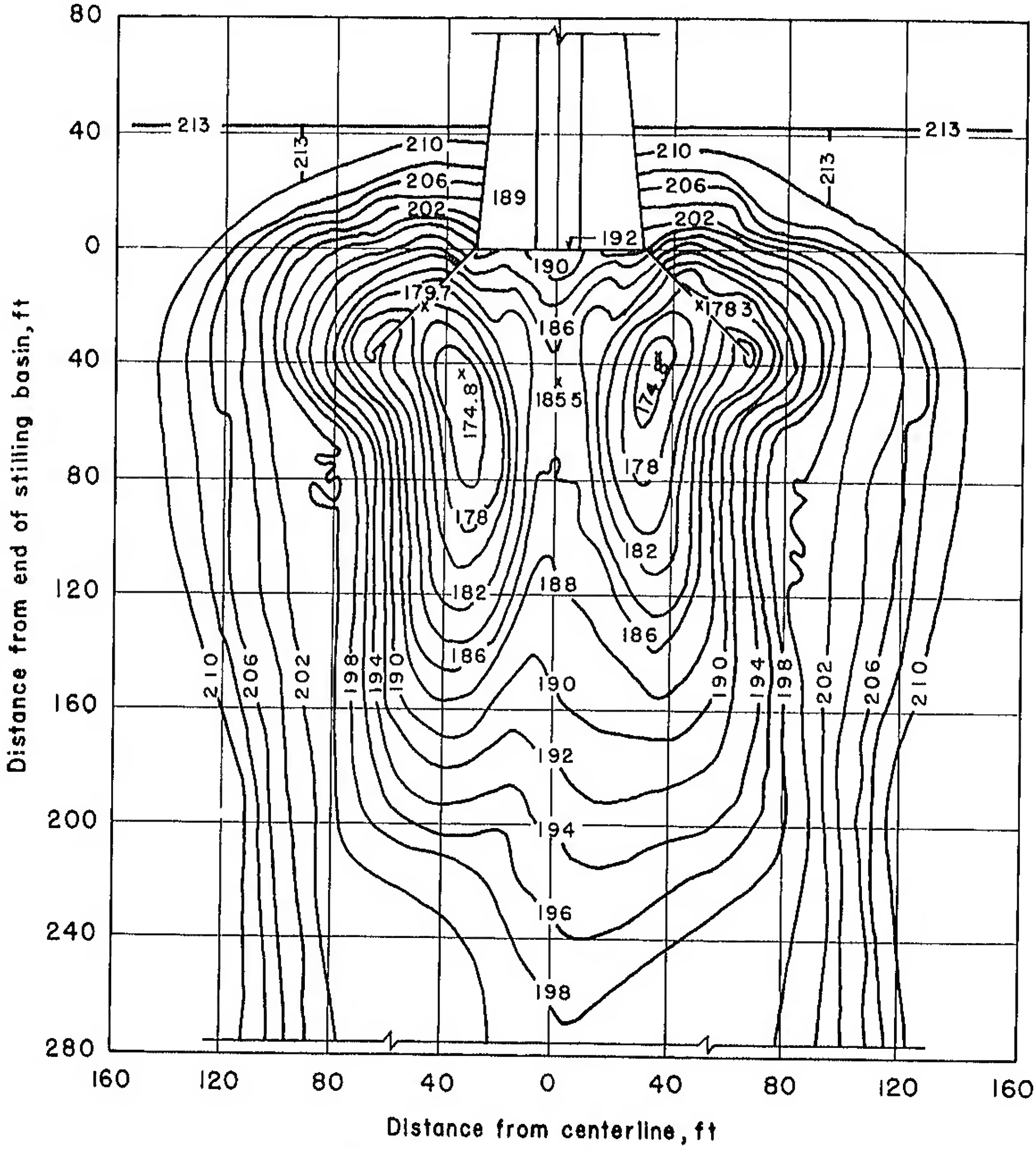


Figure 25.—Scour contours for coarse sand downstream channel after test 6.

model results to contract drawings for the field construction of the channel enlargement. Test 6, the results of which are presented in figure 25, was made to obtain the data needed to develop the geometric form of the channel enlargement.

The downstream channel test plan further assumed that the erosive forces on the periphery of the self-formed scour shape would be relatively small and approximately identical over the entire surface and that this surface could be stabilized with a relatively thin layer of small riprap.

Site Considerations

Because of the unexpectedly severe scour obtained during test 6, SCS representatives were invited to review the progress of the tests and consult with the researchers regarding possible alternatives that would meet the conditions existing at the structure site. This review and consultation was made by K. M. Hayward, Head, State Design Unit, Oxford, Miss., and Richard M. Mathews, who represented both the Fort Worth, Tex., Engineering and Watershed Planning Unit and the Washington Engineering Division office.

Several modifications to the stilling basin were tried, but none of them had a significant beneficial effect on the scour in the downstream channel. With regard to the depth of scour in the downstream channel, the pertinent part of Hayward's trip report reads:

> . . . . due to the sandy conditions that normally exist in our area and the high water tables and boils that result in deep excavation, it was felt that a scour depression of this magnitude [17 ft. below the top of the end sill] would not be acceptable even if it was moved away from the structure. The piping hazards involved in holes of this size could present a serious threat to the stability of this structure.

As a result of these considerations, the minimum acceptable elevation of the riprap surface in the downstream channel was set at elevation 186, 6 feet below the end sill elevation but still 11 feet above the minimum elevation of the scour in the self-formed downstream channel. It was recognized that heavy riprap would be required to maintain the bed at this level. These practical considerations were incorporated into the subsequent planning of the downstream channel geometric shape and the riprap protection.

Downstream Channel Geometry

To assist in designing the riprap placement geometry, two cross sections were scaled from figure 25. One cross section was taken at the end of the wingwalls, 36.7 feet downstream from the end sill where the scoured channel was widest. The other cross section was taken 93.6 feet downstream from the end sill where the contours on the lower slopes of the scoured channel were approximately parallel to each other. These scoured cross sections are shown as short dashes in figure 26.

The first step in defining the geometric cross sections at the two sections in figure 26 was to draw on figure 26 at elevation 186 the minimum elevation of the downstream channel established during the conference with the SCS designers. In figure 26, B, the width of the scoured cross section at elevation 186 is seen to be 100 feet. This established the width of the bottom riprap. Since the lower side slopes of the scoured channel slope at nearly 1 on 2, this slope was projected to the top of the channel. However, figure 26 shows that the scoured shape above about elevation 200 is flatter than 1 on 2. This encroachment on the scoured area violated the test plan assumptions and required a redesign as explained in "Test of Initial Riprap Design, Upper Banks," page 32.

The riprap section developed for figure 26, B, encroaches on the scoured section shown in figure 26, A. Because heavy riprap was required to protect the raised bed and was carried up the side slopes to elevation 192, it was hoped that the section established for figure 26, B, could be maintained at the section shown in figure 26, A, up to elevation 192. At elevation 192 the section was widened to meet the scoured bank, then continued up the scoured bank on the 1-on-2 slope.

After the preliminary geometric shapes and dimensions had been established from the cross sections, the remainder of the outlet channel geometry was developed on a copy of the scour contour map (fig. 25).

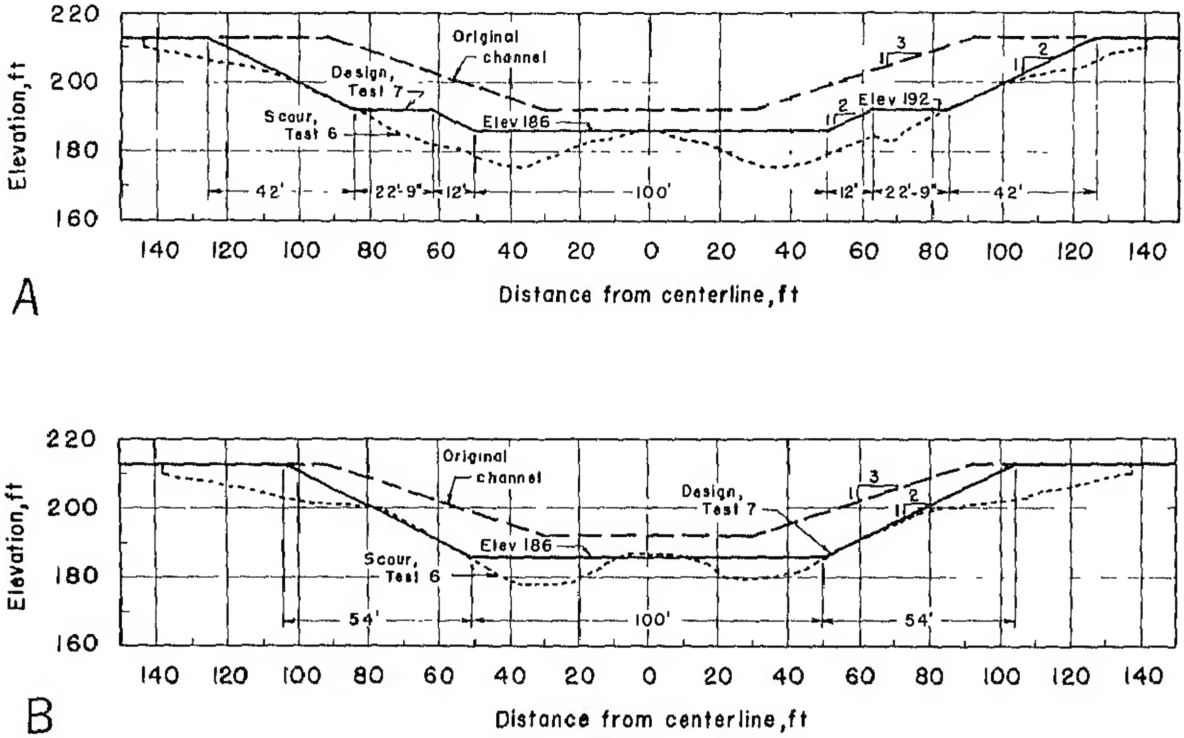


Figure 26.—Downstream channel cross sections: A, At end of wingwall, 36.7 feet downstream from end sill; B, 93.6 feet downstream from end sill.

During the consultations with the SCS designers, it was agreed that the riprap elevation at the stilling basin end sill and along the stream faces of the wingwalls would be at the end sill elevation, elevation 192. The riprap then would slope down at 1 on 2 perpendicular to the wingwalls and end sill to meet the channel bottom riprap at elevation 186. This arrangement is shown in figure 27.

Since the remainder of the downstream channel geometry for test 7 was developed from the contour map of test 6, the test 6 contours are also shown in figure 27.

The contours opposite the ends of the wingwalls approximate circular arcs centered on the ends of the wingwalls. The contour at elevation 192 has a radius of 18 feet. The contour at elevation 213 has a radius of 60 feet. The slope of the bank is 1 on 2 as shown in figures 26, A, and 27.

The downstream face of the dam has a slope of 1 on 3.

The contours between those on the dam face and the arcs centered on the ends of the wingwalls are tangent to the arcs at a 30° angle to the dam axis (60° angle to the outlet structure centerline), since this shape seemed to best fit the scour contours.

The scour contours between the arcs centered on the ends of the wingwalls and the contours parallel to the channel centerline represented by the adopted cross section shown in figure 26, B, are well represented by circular arcs of constant 18-foot radius.

The riprapped channel bed at elevation 186 was extended downstream until it met the approximate downstream limit of the scoured bed at elevation 186, 144 feet downstream from the end sill. From this point the bed was sloped upward at 1 on 10, which is a little flatter than the slope of the scoured bed, to elevation 192, the established bed elevation of the downstream channel. The 1-on-10 slope was 60 feet long and terminated 204 feet downstream from the end sill.

The 60-foot length where the bed rose from elevation 186 to elevation 192 was also a channel side-slope transition section where the channel cross section changed from the 100-foot bottom width and 1-on-2 side slopes

of the widened channel to the 60-foot bottom width and 1-on-3 side slopes of the downstream channel.

The geometric shape of the downstream channel for test 7 is shown in figure 27.

Riprap Size

The riprap used to protect the downstream channel is also shown in figure 27. Large graded riprap (D_{50} = 9 in., $D_{max.}$ = 25 in.) was placed in the channel bed and up to elevation 192 for 144 feet downstream from the end sill and around the ends of the wingwalls at elevation 192. These are areas where the established riprap geometry encroached on the scoured geometry. Small riprap (D_{50} = 3 in.,

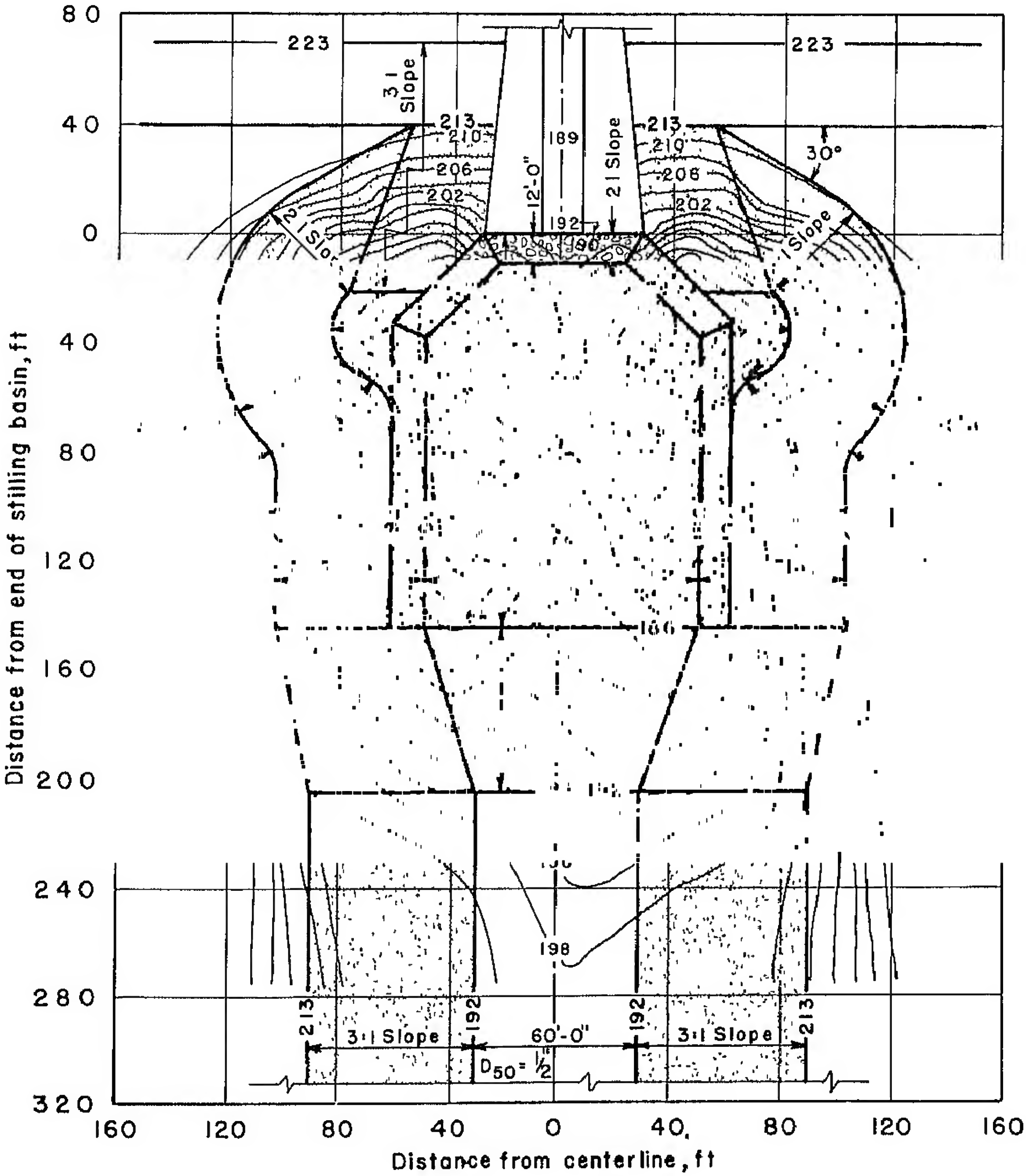


Figure 27.—Downstream channel riprap plan for test 7.

$D_{max.}$ = 6 in.) was used on the bed and banks between 144 feet and 204 feet downstream from the end sill and on all banks above elevation 192. Except near the water surface these were areas where the established downstream channel geometry approximated the scoured geometry.

The bed of the downstream channel beyond 204 feet from the end sill was formed in coarse sand (D_{50} = 0.5 in., $D_{max.}$ = 3.5 in.). The banks were covered with small riprap (D_{50} = 3 in., $D_{max.}$ = 6 in.).

The riprap sizes are shown in figure 27 and listed in table 1.

Model Channel Construction

The first step in constructing the model downstream channel was to form the channel to the designed geometric shape in coarse sand. This construction step is shown in figure 28. The next step was to remove 36 inches of sand from the areas to be covered with coarse graded riprap and 9 inches of sand from the areas to be covered with small riprap. The riprap was placed immediately after the sand was removed from each small area of the bed and banks.

The riprapped channel before test 7 is shown in figure 29. The geometric shape, the dimensions, and the riprap placement are shown in figures 27 and 28.

Test of Initial Riprap Design

The downstream channel riprapped as just described was tested by subjecting it to the design discharge (actually 11,100 c.f.s.) for 2.9 prototype days. This was test 7.

Figure 30 is a contour map of the scoured bed and figure 31 shows the contours outlined in white yarn.

Although test 7 (2.9 days) was much shorter than test 6 (11.2 days), its length was sufficient to pinpoint the inadequacies of the initial riprap design and to presumably show where the initial riprap design provided adequate protection.

Bed Scour

Figures 30 and 31 show that riprap was removed from the bed between the wingwalls and that the deepest scour is approximately in line with the stilling basin sidewalls. The deepest scour of the sand bed also occurred in these approximate locations. From the right side 2.3 feet of riprap—almost the entire 3-foot

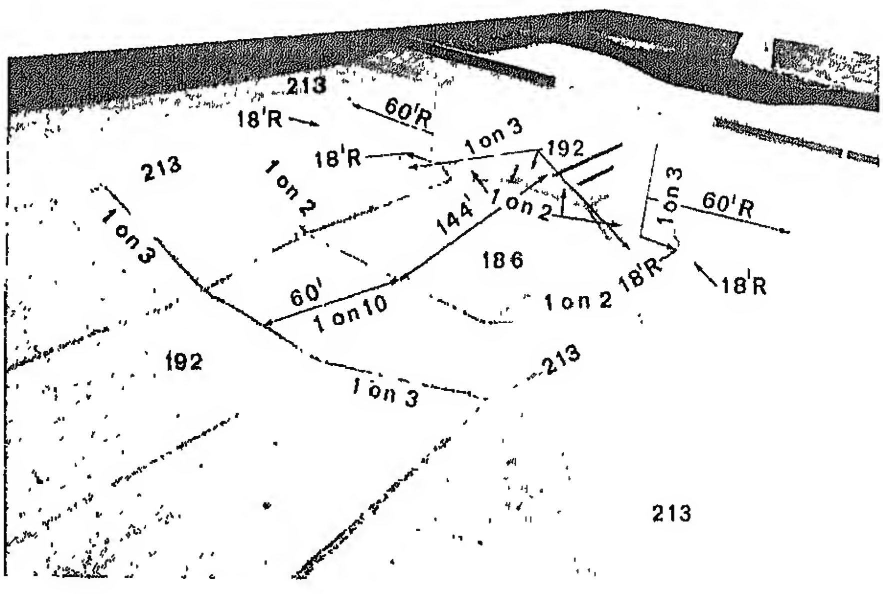


Figure 28.—Downstream channel geometric shape formed in coarse sand before placing riprap for test 7. Marks in sand delineate boundaries of various surfaces.

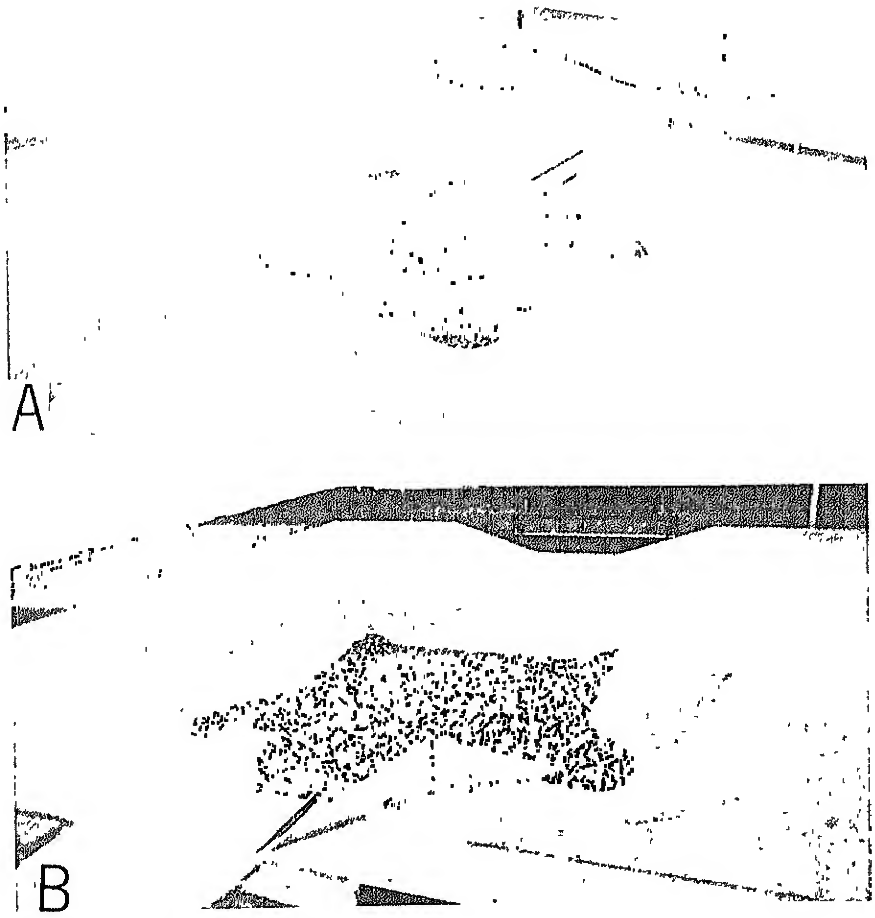


Figure 29.—Shaped and riprapped downstream channel before test 7 showing views from downstream (A) and from upstream (B).

thickness of riprap placed there—was scoured. Less riprap—1.1-foot maximum depth—was removed from the left side. Some riprap was removed downstream from the end sill and along the right wingwall. Apparently much of the eroded riprap was deposited along the channel centerline. It is apparent that heavier riprap will be required to protect the bed between the wingwalls.

Raising the riprap for test 7 into the hole scoured in the sand bed of test 6 decreased the volume of water available for energy dissipation and changed the flow pattern in the downstream channel. As a result, the small riprap and the sand bed downstream of the heavy riprap were scoured to a depth of 4.4 feet below elevation 186. Also, a comparison of figures 25 and 30 shows that the elevation 186 contour moved about 60 feet farther downstream during test 7. It is obvious that the heavy riprap must extend farther downstream.

Lower Banks

Fine sand has been deposited along the sides of the channel over the heavy riprap and over the lower banks that were protected by small riprap. Around the ends of the wingwalls fine sand has also covered much of the level surface at elevation 192. Possibily these sand deposits would have been rearranged if the test had been continued for a longer period. When the deposits were removed, there was no evidence that the riprap under them had moved.

Beside Stilling Basin

The small riprap upstream of the ends of the wingwalls was undisturbed except for a small amount of erosion caused by wave wash

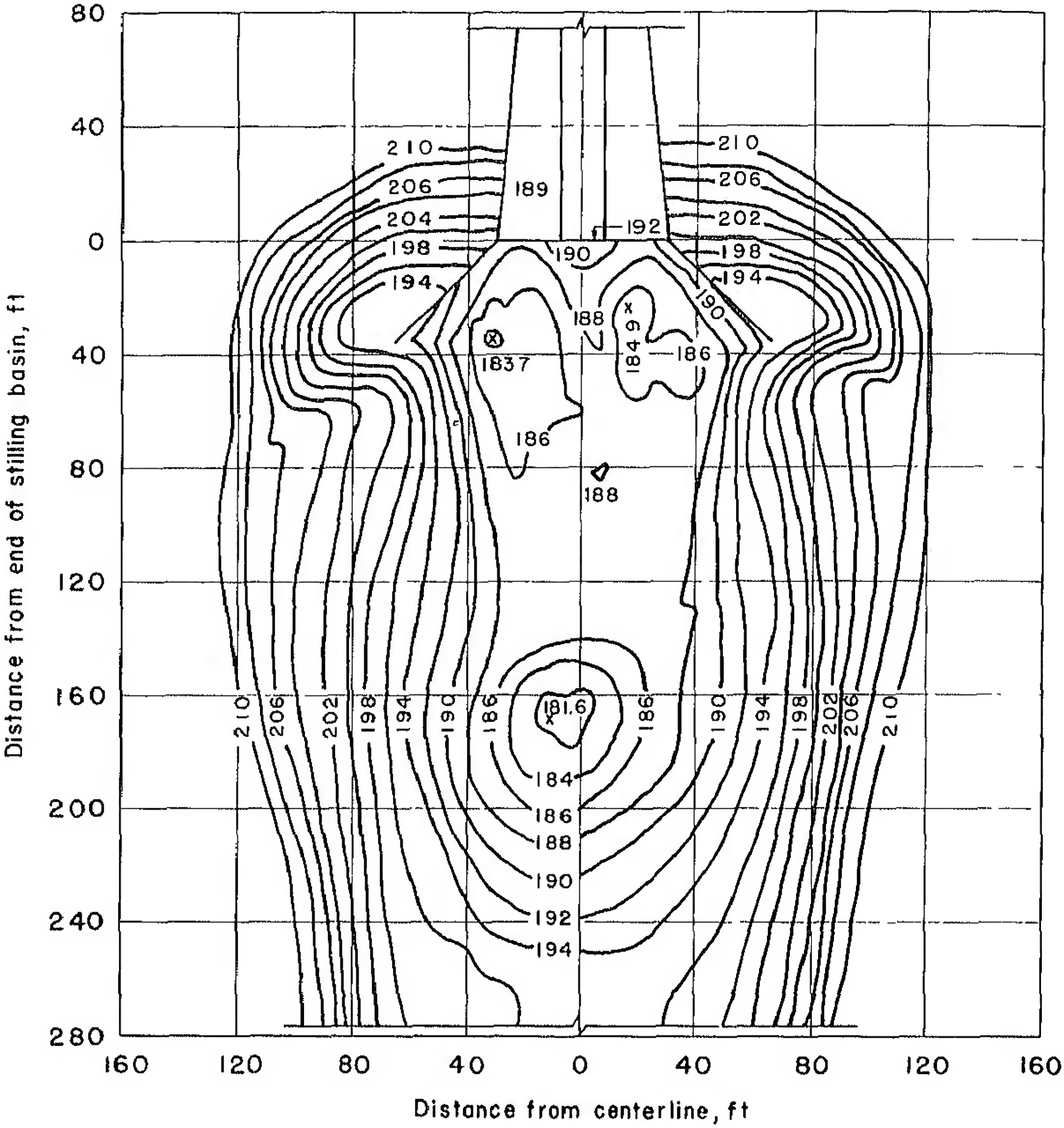


Figure 30.—Scour contours for initial downstream channel riprap design after test 7.

between elevations 206 and 210. Test 7 shows that the small riprap protection in this area was adequate except for a short distance upstream of the wingwall ends near the water surface.

Upper Banks

It is noted in the preceding description of the "Initial Riprap Placement Design" (p. 26) that the scoured side slopes above about elevation 200 were flatter than 1 on 2 and that the designed side slopes of 1 on 2 encroached on the area scoured during test 6. The result of this encroachment is shown in figures 30 and 31. The small riprap was too small and the tops of the banks were severely eroded. This is the source of the sand deposited on the lower slopes. Obviously the tops of the banks need reshaping to prevent encroachment on the areas scoured during test 6 if small riprap is to be used to stabilize the banks. An alternate solution is the use of heavier riprap.

Redesign of Riprap Placement

The only change in the geometry of the riprap placement from test 7 was to extend farther downstream the trapezoidal cross section with a bottom width of 100 feet at elevation 186 and side slopes of 1 on 2. This cross section for test 7 was terminated 144 feet downstream of the end sill. For test 8 this distance was increased to 215 feet. The new dimensions are shown in figure 32.

Several changes were made in the riprap placement. The riprap properties mentioned in the following discussion are given in table 1, page 8.

Large uniform riprap (D_{50} = 16 in., $D_{max.}$ = 23 in.) replaced the large graded riprap for 60 feet downstream from the end sill to protect the areas of the bed and the slopes along the end sill and wingwalls that had scoured during test 7. A filter layer of large graded riprap was used under the large uniform riprap.

Large graded riprap (D_{50} = 9 in., $D_{max.}$ = 25 in.) replaced the small riprap previously used in the transition section between the 100-foot wide 1-on-2 side-slope section and the 60-foot wide 1-on-3 side-slope downstream channel.

For two reasons large graded riprap replaced the small riprap on the banks downstream of

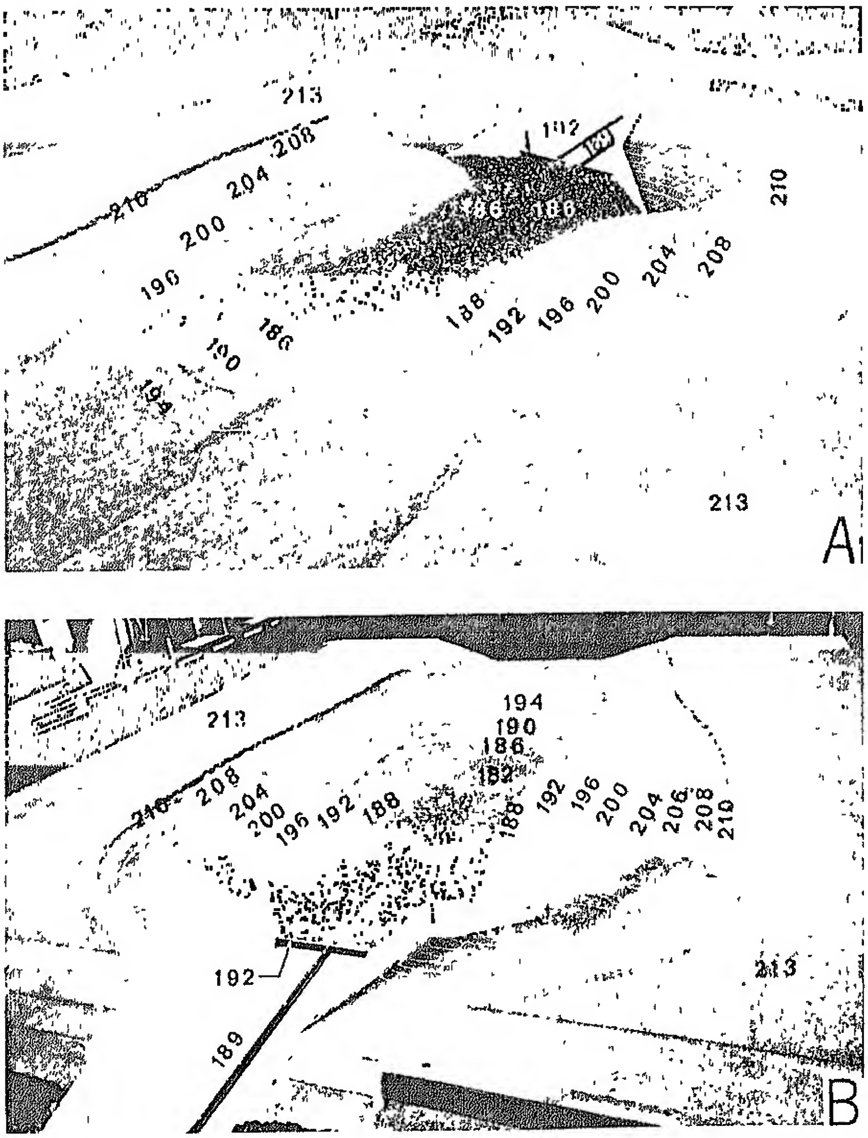


Figure 31.—Appearance of initial downstream channel riprap design after test 7 showing views from downstream (A) and from upstream (B). (Design discharge 11,100 c.f.s. for 2.9 days.)

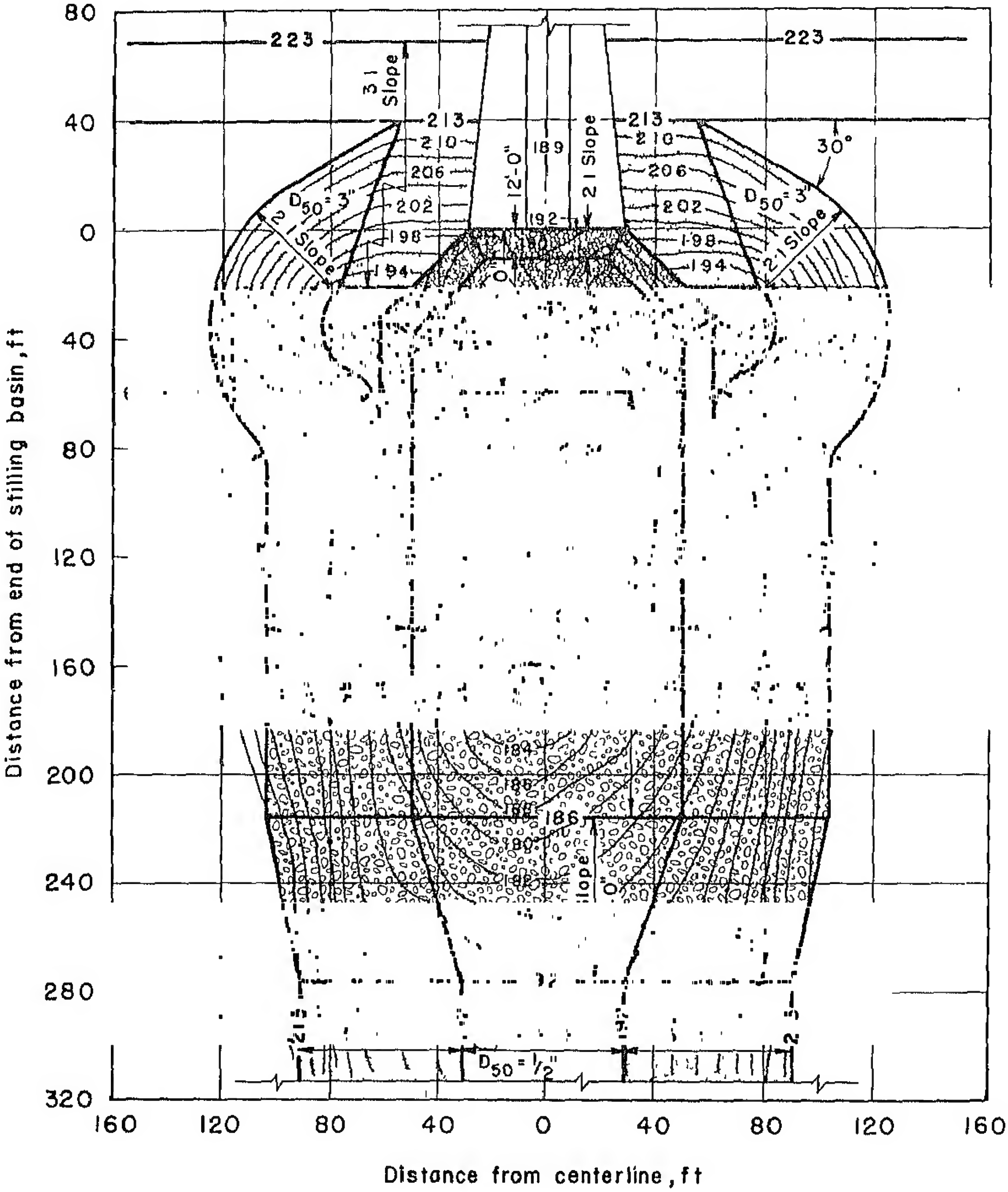


Figure 32.—Downstream channel riprap plan for test 8.

the cross section 21 feet downstream of the end sill defined by the elevation 192 contour on the dam face. First, riprap heavier than the small riprap used for test 7 was required to prevent scour above about elevation 200 where the 1-on-2 bank slopes encroached on the flatter slopes scoured in the sand used for test 6. And second, since the prototype tailwater elevation could not be determined with certainty, the lower elevation of the heavy riprap also could not be determined with certainty. Therefore the small riprap, which provided

adequate protection between elevations 192 and 200 during test 7, was replaced with coarse graded riprap for test 8.

The banks of the downstream channel were protected with small riprap. The downstream channel bed was formed in coarse sand.

Test of Redesigned Riprap

For test 8 the riprap plan described in the previous section and shown in figure 32 was installed in the model. The design flow (11,200 c.f.s.) and the design tailwater level (elevation 209.0) were established. At the end of 4.24 prototype hours (1 model hr.) the flow was stopped so the performance could be observed. Following the visual observations the design conditions were reestablished for a total test time of 3.8 days.

Initial Visual Observations

During the flow there was a strong upstream current along the sides of the channel from 130 to 94 feet downstream of the end sill, but there was no scour of the coarse graded riprap bank protection. Also during the flow the appearance of the turbulence indicated that the length of the bed riprap protection was about correct. The surface currents are shown in figure 33.

Visual observations after the flow was turned off showed no movement of the heavy riprap. There had been some movement of the small riprap at the water surface beginning 21 feet downstream of the end sill, with decreasing erosion to points opposite the end sill. The sand bed in the downstream channel had degraded about 1 foot during this initial test period.

Test Results

A contour map of the downstream channel after test 8 is presented as figure 34. Figures 35 and 36 are "before" and "after" photographs. All the "before" and "after" contours coincide except in three areas:

(1) *Beaching Opposite Stilling Basin.*—Although hardly apparent in figures 34 and 36, there is beaching of the small riprap at the waterline for a distance of 21 feet downstream from the end sill. These areas have been crosshatched in figure 34. The erosion did not penetrate the small riprap. In fact even after 3 months had elapsed, during which the tests were completed and the model had been demonstrated many times, the small riprap had been barely penetrated.

Figure 33.—Confetti shows surface currents and turbulence in downstream channel. Final riprap arrangement as used for test 8. (Discharge 11,865 c.f.s., exposure 0.42 sec.)

(2) *Bed Riprap Movement.*—The second area of riprap movement is in the channel bed just downstream of the large uniform riprap. There the large graded riprap was scoured to a maximum depth of 1.8 feet. This scour could be prevented by extending the large uniform riprap an additional 30 feet. However, it seems likely that applying a safety factor to the barely adequate riprap size used in the model will result in riprap that would not move in the prototype.

(3) *Downstream Channel Scour.*—The third area of scour is in the downstream channel just downstream of the large graded riprap. The unprotected coarse sand bed scoured a maximum of 5.6 feet. The banks, protected by small riprap, also eroded. The erosion on the left bank is below elevation 198 and on the right bank is below elevation 201. It appears that the toe of the bank riprap was exposed by the bed degradation and that the small riprap progressively raveled from the toe of the slope. Although most of the bank protection was lost during the following months of tests and demonstrations, it appears that the small riprap

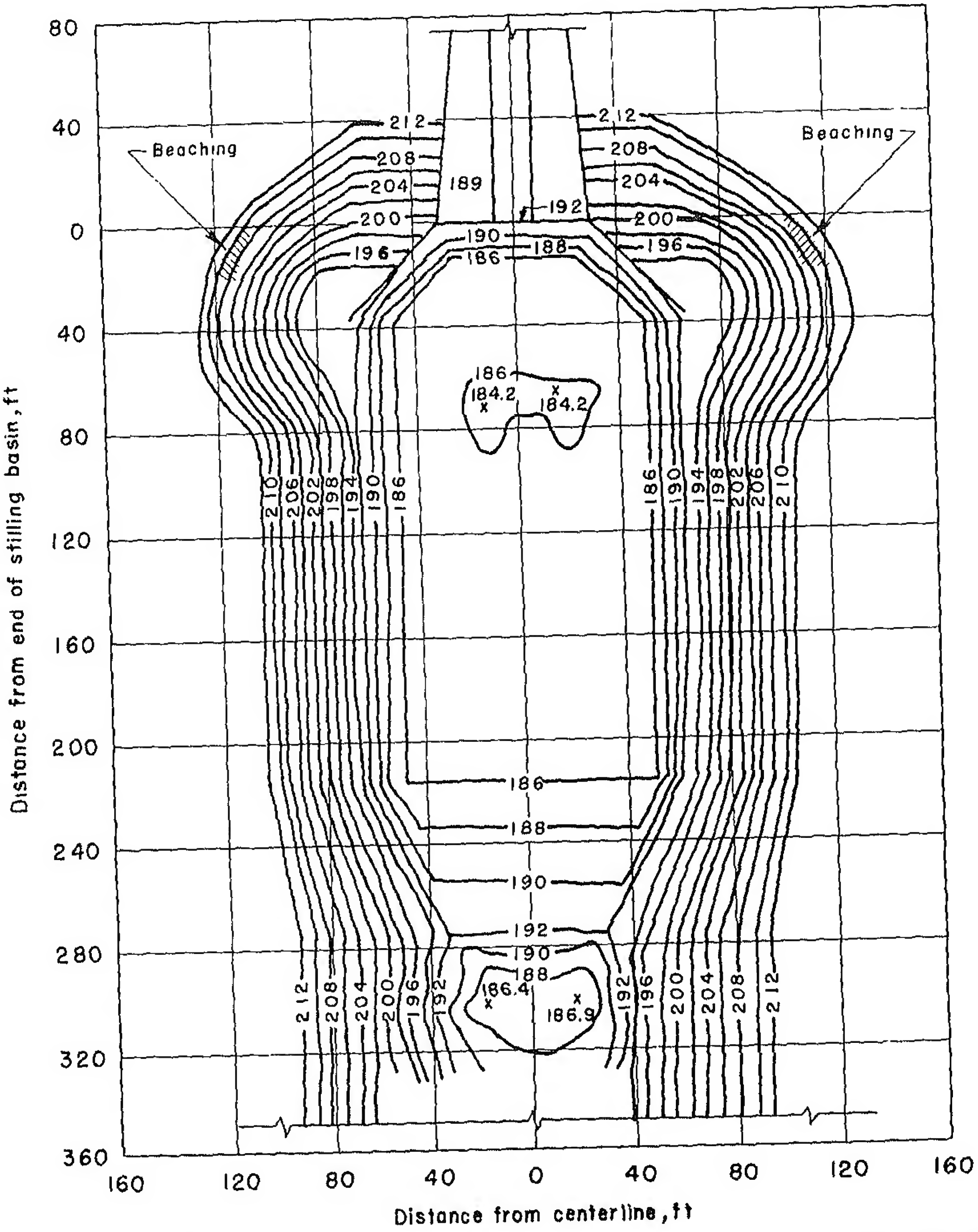


Figure 34.—Downstream channel contour map after subjecting redesigned riprap plan to design discharge for 3.8 days (test 8).

would have protected the downstream channel banks had raveling of its toe been prevented.

Adequacy of Downstream Channel Bed Material Size.—An interesting check of the ability of the downstream channel to resist scour is possible through the use of the criteria devel-

oped by Anderson et al.[7] Model dimensions will be used to make this analysis.

The model trapezoidal downstream channel has a bottom width of 3.33 feet, a flow depth of 0.94 foot, and 1-on-3 side slopes. The design discharge is 8.17 c.f.s. For the flow area of 5.82 square feet, the average velocity is 1.40 f.p.s.

For design purposes the average velocity must be increased because the bed shear is not uniformly distributed across the channel. Using Anderson et al. symbols, $B/y = 3.33/0.94 = 3.53$ and $Z = 3$. From their figure 11 the maximum boundary shear stress on the sides is about 1.3 times and from their figure 12 the corresponding figure for the bed is 1.5 times the average shear stress. The maximum shear stress is therefore on the bed and is 1.5 times the average shear stress. The velocity for use in determining stable riprap size is therefore $1.40\sqrt{1.5} = 1.71$ f.p.s. When this velocity and the depth of flow are inserted into equation 17, page 23,

$$D_{50} = 0.00116 \frac{V^3}{\sqrt{d}} = 0.00116 \frac{(1.71)^3}{\sqrt{0.94}} \quad (18)$$

$$= 0.00568 \text{ foot or } 1.73 \text{ mm.}$$

[7] See footnote 4.

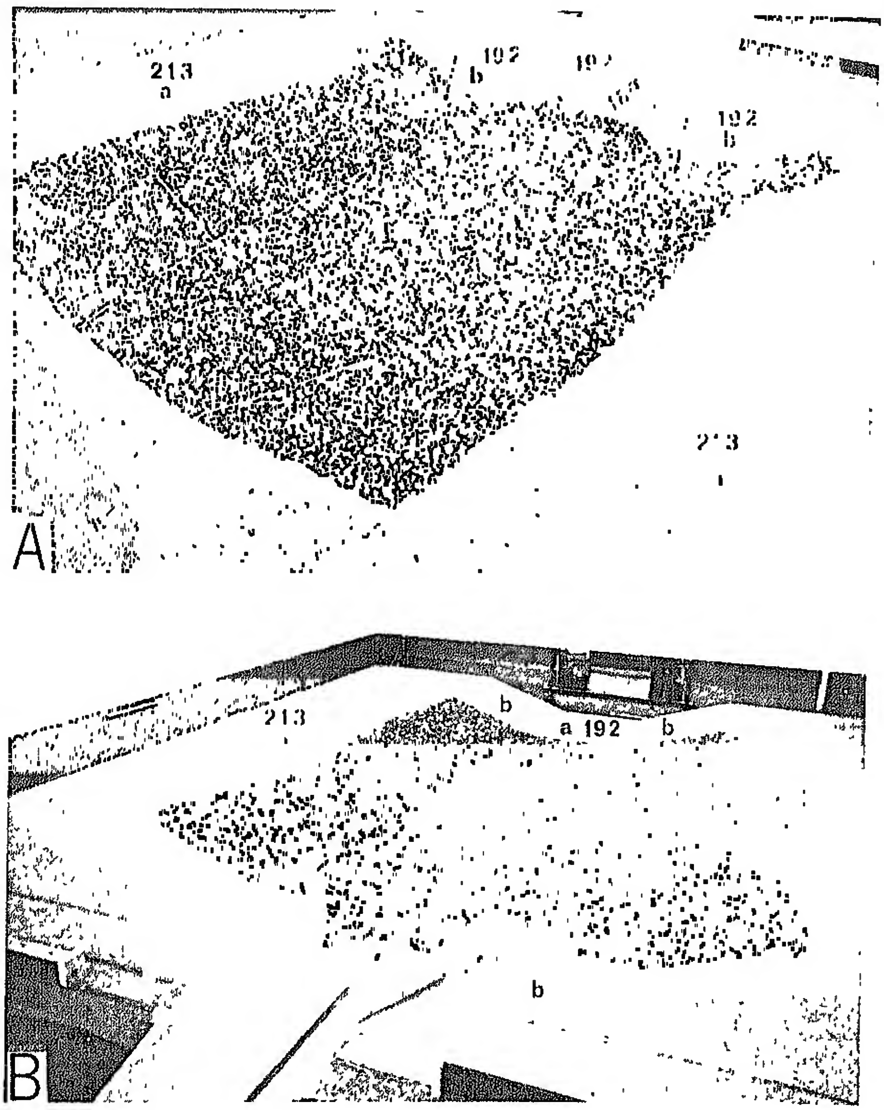
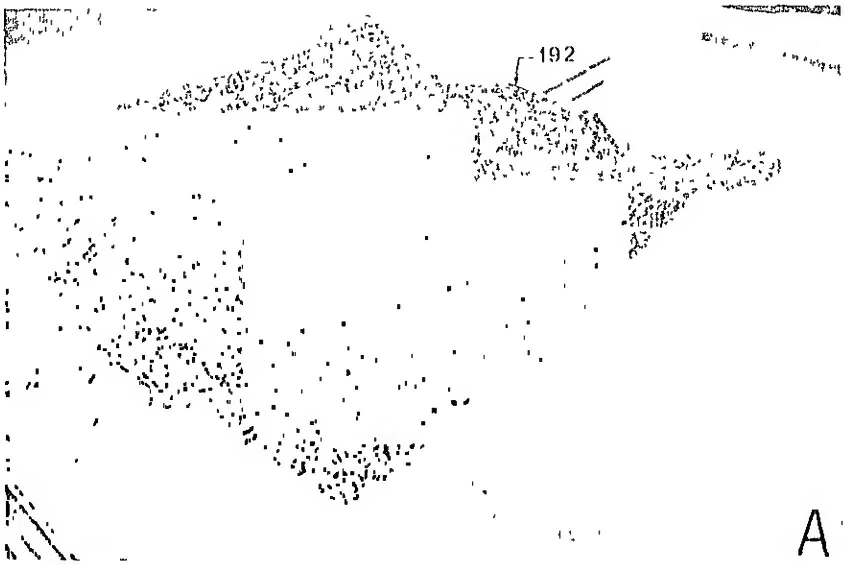


Figure 35.—Appearance of redesigned riprap installed in downstream channel for test 8 showing views from downstream (A) and from upstream (B). Yarn shows boundaries of the geometric surfaces. Bed materials are coarse sand (a), small riprap (b), large graded riprap (c), and large uniform riprap (d).



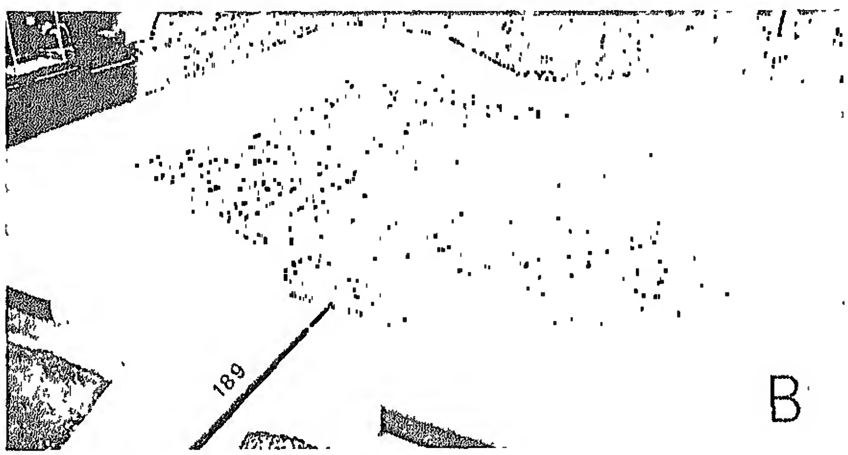


Figure 36.—Appearance of redesigned riprap installed in downstream channel after 3.8 days at design discharge (test 8) showing views from downstream (A) and from upstream (B). There is a little beaching of the small riprap between the elevation 208.0 and 210.0 contours upstream of the large riprap and opposite the wingwalls, some scour of the coarse graded riprap downstream of the coarse uniform riprap that is outlined by elevation 186.0 contour in center of channel, and scour of the channel bed and lower banks at downstream end. Otherwise these "after" contours also represent the "before" contours.

For the coarse sand used in the model downstream channel bed, D_{50} = 0.70 mm. It is obvious that the downstream channel bed will erode. For the small riprap used on the banks, D_{50} = 4.25 mm. This is more than adequate to protect the banks. These computations agree with the observations. However, a point to remember is that the computations assume normal velocity distribution in the trapezoidal channel; and it is probable that normal velocity distribution had not been established at the point where the observations were made in the model downstream channel.

Recommendations

Because the tests of the riprap plan used for test 8 yielded results considered generally satisfactory, the downstream channel geometry, the riprap placement, and the riprap sizes (increased by a safety factor) shown in figure 32 are recommended.

STILLING BASIN SWEEPOUT TESTS

The stilling basin sweepout tests were a brief series of observations conducted as test 12, the last test made. They totaled about 30 minutes. Each test was conducted by successively lowering the tailwater level. Observations consisted of the location of the toe of the standing wave in the stilling basin and movement of the riprap.

Tailwater Level Design

The stilling basin was designed by C. T. Myers of the Mississippi Design Unit in January 1970 using the criteria established for the hydraulic design of the box inlet drop spillway.[8] A review of Myers' computations shows that the design tailwater elevation, elevation 209.0, is 3.3 feet higher than the minimum tailwater elevation established by the design criteria, elevation 205.7. This difference should be kept in mind when evaluating the observations.

Observations

Tailwater elevation 208.9 (design elevation is 209.0)—The toe of the standing wave was at the beginning of the stilling basin sidewall flare, 82 feet upstream from the end of the stilling basin.

Tailwater elevation 207.7 (1.3 ft. below the design elevation; 2.0 ft. above the design criteria elevation)—There was no change in the position of the standing wave.

Tailwater elevation 206.6 (2.4 ft. below the design elevation; 0.9 ft. above the design criteria elevation)—The toe of the standing wave was 77 feet upstream from the end of the stilling basin. There was a considerable drop in the water surface as it passed over the end sill, and there was a standing wave downstream of the end sill.

Tailwater elevation 205.6 (3.4 ft. below the design elevation; 0.1 ft. below the design criteria elevation)—The toe of the standing wave was perpendicular to the centerline and 48 feet upstream from the end of the stilling basin.

Tailwater elevation 204.6 (4.4 ft. below the design elevation; 1.1 ft. below the design criteria elevation)—The toe of the standing wave was perpendicular to the centerline and 37 feet upstream from the end of the stilling basin. There was no movement of the large riprap on the banks of the enlarged downstream channel nor of the small riprap beside the stilling basin. The downstream channel banks were eroding.

Tailwater elevation 202.5 (6.5 ft. below the design elevation; 2.2 ft. below the design criteria elevation)—The toe of the standing wave was perpendicular to the centerline and 19 feet upstream from the end of the stilling basin. There was a standing wave over the end sill, and there was another standing wave about opposite the downstream end of the wingwalls.

Comments

At the end of this short but nevertheless severe test of the downstream channel riprap design, there had been no significant movement of the large riprap except just downstream from the ends of the stilling basin end sill. There a few stones had been displaced.

That the tailwater level be below the design level is certainly not recommended. Especially not recommended is lowering the tailwater below the elevation established by the design criteria. However, these brief tests of stilling basin sweepout show that little damage occurred to the riprap-protected downstream channel.

[8] Blaisdell, F. W., and Donnelly, C. A. Hydraulic design of the box-inlet drop spillway. U.S. Dept. Agr. Agr. Handb. 301, 40 pp. 1966.

CAPACITY OF COMPOUND TRAPEZOIDAL WEIR BOX INLET DROP SPILLWAY

The relationship between the flow and the water surface elevation for the compound trapezoidal weir box inlet drop spillway was determined by running known flows through the spillway and measuring the elevation of the upstream water surface. The experimental data are plotted in figure 37. The relationship was subsequently expressed in the form of equations to broaden its usefulness, and the discharge coefficients in the equations were evaluated. A check of the discharges computed from equations against the test discharge shows the agreement to be within 6 percent.

Compound Trapezoidal Weir

The original shape of the weir for the box inlet drop spillway modeled and shown in figures 1 and 3 is considered to be a trapezoid because, when the weir is developed in elevation, its shape is that of a trapezoid having a base width of 58 feet and 1-on-3 end slopes. The base width is the sum of the box inlet width (42 ft.) plus the lengths of the two longitudinal horizontal sections of the crest at elevation 204 (2 x 8 ft.). The end slopes coincide with the slope of the upstream face of the dam.

The compound trapezoidal weir is shown in figure 6 and the developed weir on which tests were made is shown in figure 38. This weir shape is called a compound trapezoid because there is a low level trapezoid between elevations 204 and 213, and the section above elevation 213 is considered to be a second trapezoid. Both the design prototype dimensions and the actual dimensions scaled up from the model are given in figure 38. The actual dimensions, shown in parentheses in figure 38, were used in the computations and apply to the experimental data presented in table 2 and in figure 37.

Test Data

The data obtained during test 9, the calibration test, are presented in table 2. They are plotted in figure 37 to show the relationship

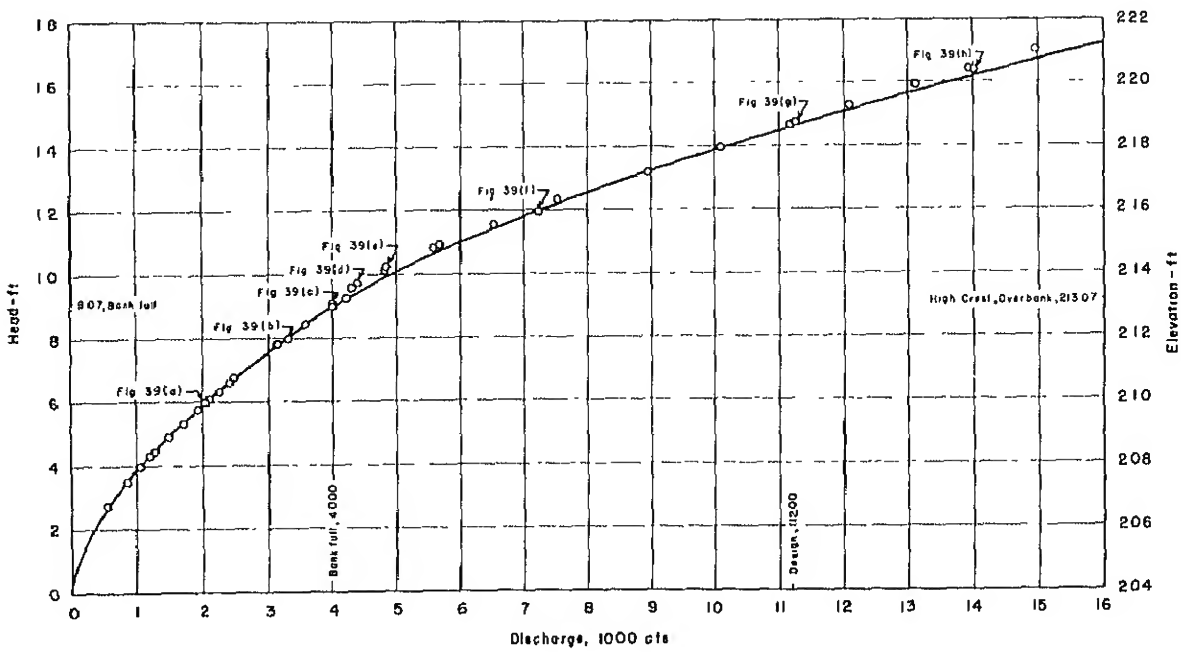


Figure 37.—Head-discharge relationship for compound trapezoidal weir box inlet drop spillway.

Table 2.—Head-discharge data and comparisons

Run No.	Elevation	Head on weir (H)	Discharge (Q)	Computed discharge (Q_C)	$\frac{Q_C - Q}{Q}$
	Feet	*Feet*	*C.f.s.*	*C.f.s.*	*Percent*
		Channel flow			
0	204.00	0	0	...	...
1	208.46	4.46	1,274	1,281	+0.5
2	210.77	6.77	2,494	2,498	+ .2
3	211.81	7.81	3,140	3,152	+ .4
4	212.44	8.44	3,582	3,580	- .1
5[1]	213.09	9.09	4,003	4,047	+1.1
18	210.62	6.62	2,414	2,409	- .2
19	210.34	6.34	2,263	2,247	- .7
20	210.08	6.08	2,112	2,100	- .6
21	209.74	5.74	1,916	1,915	- .1
22	209.35	5.35	1,718	1,711	- .4
23	208.91	4.91	1,487	1,492	+ .3
24	208.32	4.32	1,207	1,218	+ .9
25	207.49	3.49	852	871	+2.2
26	206.74	2.74	563	597	+6.0
28	207.98	3.98	1,073	1,070	- .3
29	209.99	5.99	2,057	2,050	- .3
30	211.97	7.97	3,313	3,259	-1.6
31	213.05	9.05	4,003	4,017	+ .3
		Overbank flow			
5[1]	213.09	9.09	4,003	4,052	+1.2
6[2]	213.59	9.59	4,281	4,486	+4.8
7[3]	214.12	10.12	4,838	5,010	+3.6
8	214.82	10.82	5,611	5,780	+3.0
9	215.56	11.56	6,525	6,680	+2.4
10	216.38	12.38	7,522	7,771	+3.3
11	217.18	13.18	8,961	8,927	- .4
12	217.95	13.95	10,114	10,122	+ .1
13	218.69	14.69	11,198	11,345	+1.3
14	219.28	15.28	12,080	12,372	+2.4
15	219.91	15.91	13,114	13,519	+3.1
16	220.40	16.40	13,953	14,447	+3.5
17	221.03	17.03	14,955	15,686	+4.9
27[4]	213.22	9.22	4,207	4,157	-1.2
32	213.74	9.74	4,377	4,628	+5.7
33	214.22	10.22	4,850	5,115	+5.5
34	214.94	10.94	5,700	5,921	+3.9
35	215.93	11.93	7,241	7,161	-1.1
36	218.72	14.72	11,247	11,396	+1.3
37	220.34	16.34	14,019	14,332	+2.2

[1] Flow just overbank.
[2] No flow over high crest.
[3] Flow over high crest.
[4] Flow on for 3.67 prototype days.

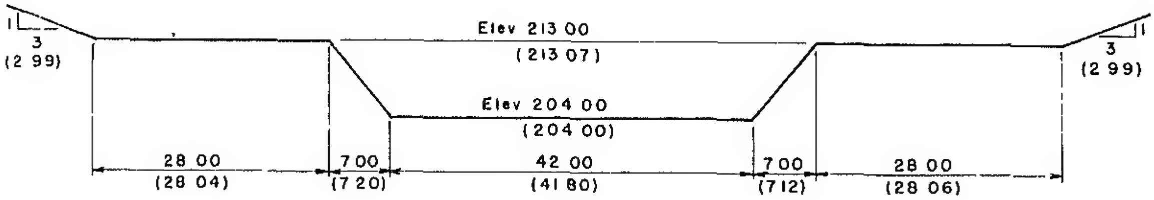


Figure 38.—Developed compound trapezoidal weir tested.

between the head or upstream water surface elevation and the discharge.

Photographs of the flow approaching the box inlet are shown in figure 39.

In figures 39, A and B, the flow is entirely within the channel. The confetti shows little movement beside the box inlet since there is no flow over the sides. All the flow enters over the upstream crest. The lack of movement of the confetti along the sides of the channel shows that the flow separates from the sides of the channel upstream of the structure. Much of this separation is caused by the almost stagnant water beside the box inlet, but part of the separation may result from the short approach channel in the model—the converging channel entrance is just outside the picture.

In figure 39, C, the approach channel is bank full. There is a shallow depth of water on the overbank area, but the lack of movement of the confetti shows that its velocity is negligible. There is no flow over the horizontal side weirs.

In figure 39, D, there is overbank flow and a change in the surface velocity where the overbank flow enters the channel. Little water is passing over the side weirs and the surface currents beside the box inlet are still negligible.

In figure 39, E, the change in the surface currents where the overbank flow enters the channel is hardly detectable. The increase in water surface elevation between figures 39, D, and 39, E, is only 0.5 foot, yet there is a considerable change in the surface currents. Although there is some flow over the side weirs in figure 39, E, the flow is not sufficient to create a significant surface current approaching the side weirs.

Surface velocities beside the box inlet are higher in figure 39, F.

The design flow is shown in figure 39, G. The surface currents are now approaching the weir in an almost perpendicular direction.

The peak runoff flow is shown in figure 39, H. The spillway has ample capacity to pass this flow, which exceeds the design flow by 25 percent.

During run 27 the flow was left on overnight, a duration corresponding to 3.7 prototype days. The flow was slightly overbank; the overbank depth was 0.22 foot. In the morning a surging—a surface oscillation—was observed. The surface currents are shown by confetti in figure 40, A. These currents eroded the face of the dam as shown in figure 40, B. The currents were outward along the dam face. The flow returned to the channel upstream of the point where it left the channel. As a result, some of the material that was eroded from the dam face was deposited on the overbank area, and some of the sand was carried through the structure. A deposit can be seen on the channel centerline just downstream from the stilling basin end sill. It is not known whether the oscillation of the water surface represents a prototype condition or if it is a model phenomenon only. It is mentioned because it was observed during the tests.

Head-Discharge Equations

The general forms of the head-discharge equations will be developed for the compound trapezoidal weir, the coefficients will be evaluated from the data, then the agreement of the equations will be tested.

Development of the Equations

Two equations will be developed. One equation will apply to the low weir—the trapezoid formed, for the modeled structure, between elevations 204 and 213; the second equation will give the discharge for the compound

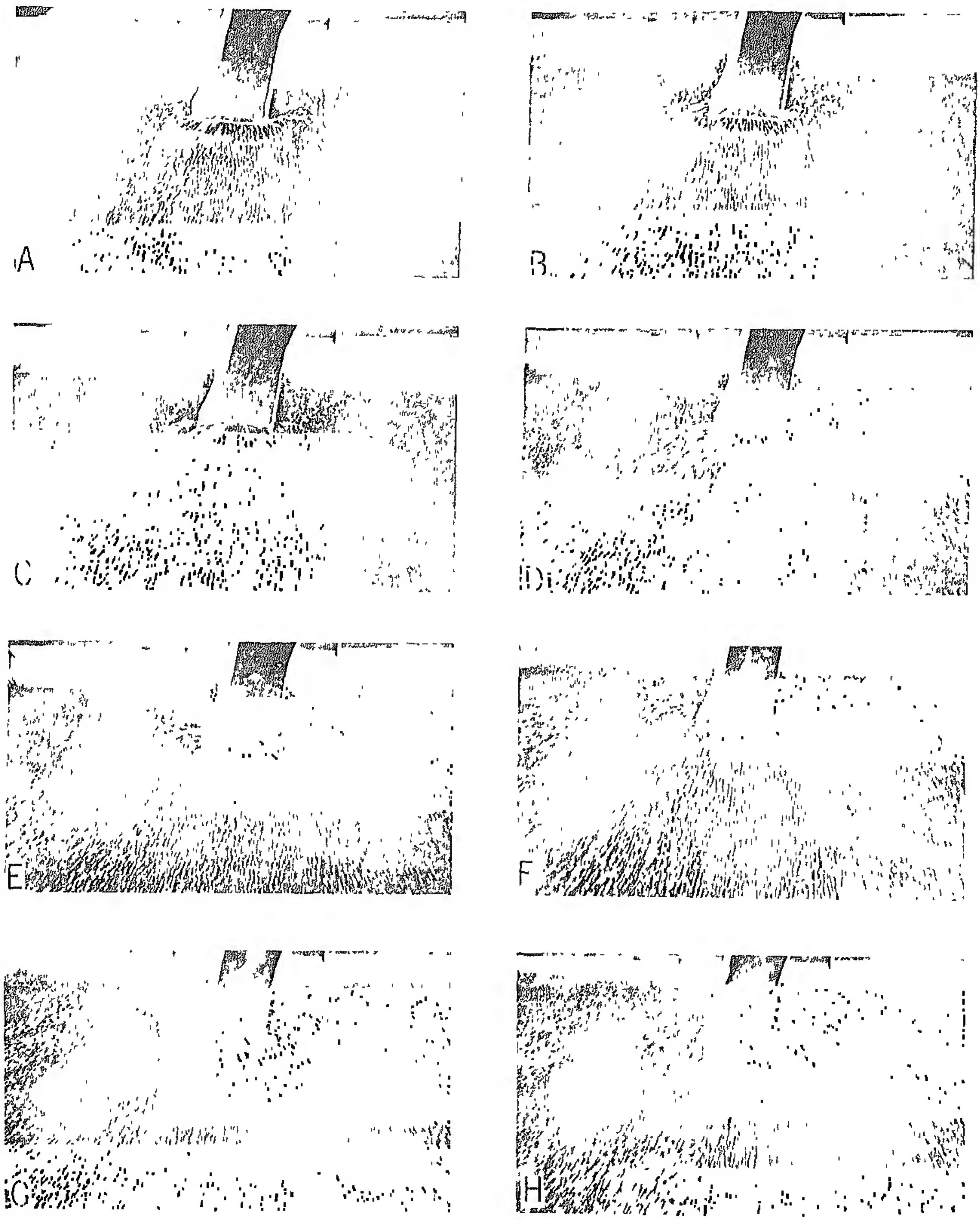
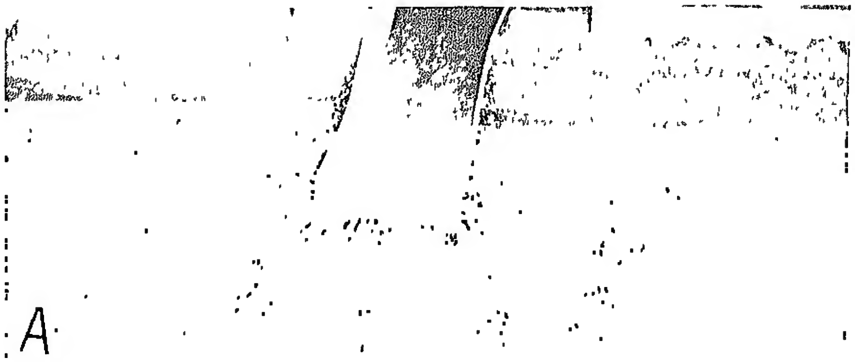


Figure 39.—Confetti shows surface currents near box inlet during test 9 (upstream crest elevation 204.00 ft.; sidewall crest elevation 213.00 ft.; exposure 0.42 sec.): A, Run 29; B, run 30; C, bank-full flow, run 31; D, low overbank flow, run 32; E, run 33; F, run 35; G, design discharge, run 36; and H, peak design runoff, run 37. [For headwater elevation and discharge for each run, see table 2.]



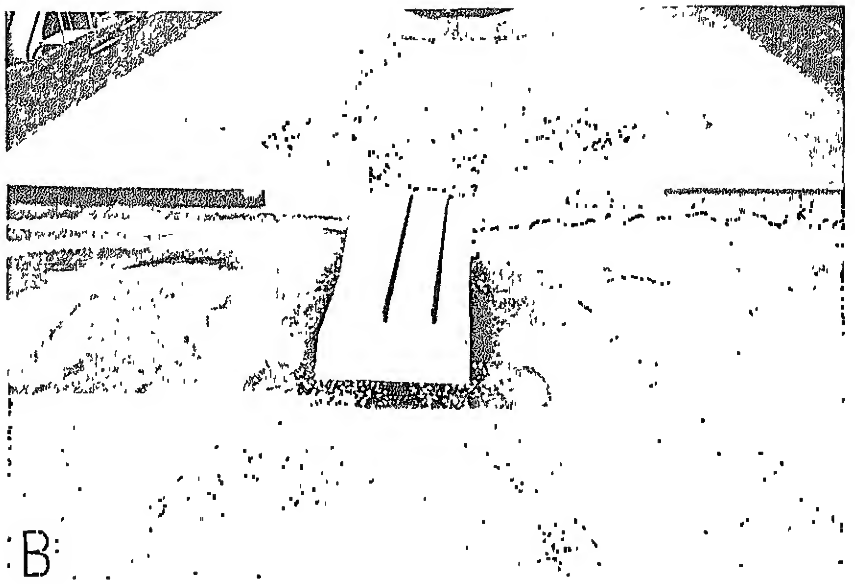


Figure 40.—Water surface oscillations during test 9, run 27, eroded dam face: A, Confetti shows surface currents; B, scour of dam face, deposits on overbank, and deposits in downstream channel. (Discharge 4,207 c.f.s. for 3.67 days; headwater elevation 213.22 ft.)

trapezoidal weir, which controls the head-discharge relationship when, for the modeled structure, the water surface elevation exceeds elevation 213.

Low Weir.—The low trapezoidal weir and the dimensional symbols are shown in figure 41, A. The weir has a base length L_L and end slopes of 1 on Z_L. The head on the weir is H. The weir is broken down into two components to compute the discharge over the weir—a rectangular weir of length L_L and a triangular weir having end slopes of 1 on Z_L. If the central angle of the triangular weir is θ, then $\tan \theta/2 = Z_L$. The sum of the discharges through the rectangular and triangular weir sections is

$$Q = C_{RL}\frac{2}{3}\sqrt{2g}L_L H^{3/2} + C_{TL}\frac{8}{15}\sqrt{2g}Z_L H^{5/2} \tag{19}$$

where C_{RL} and C_{TL} are discharge coefficients for the lower rectangular and triangular weirs, respectively, and g is the acceleration due to gravity.

Compound Weir.—The compound trapezoidal weir and its dimensional symbols are shown in figure 41, B. As before, the low weir has a base length of L_L and end slopes of 1 on Z_L. But now the low weir has a depth defined as d. The crosshatched areas, formerly treated as a triangular weir, are now joined together and treated as a triangular orifice. To the flow

through the low rectangular weir and the triangular orifice is added the flow over a high rectangular weir of length L_H and a high triangular weir having 1 on Z_H end slopes. The resulting discharge equation for the compound trapezoidal weir is

$$Q = C_{RL}\frac{2}{3}\sqrt{2g}L_L H^{3/2} + C_O\frac{8}{15}Z_L d^2\sqrt{2g}H^{1/2}$$

$$+ C_{RH}\frac{2}{3}\sqrt{2g}L_H(H-d)^{3/2}$$

$$+ C_{TH}\frac{8}{15}Z_H\sqrt{2g}(H-d)^{5/2} \qquad (20)$$

where C_{RH} and C_{TH} are discharge coefficients for the high rectangular and triangular weirs, respectively, and C_O is the orifice discharge coefficient. The other symbols have been defined previously.

Evaluation of the Discharge Coefficients

The discharge coefficients for the low rectangular and triangular weirs will be evaluated in a single step. This evaluation will be made using the head-discharge data applicable to the low weir. The rectangular and triangular weir coefficients for the high weir are assumed to be identical to those for the low weir. The validity of this and the other assumptions will be checked by comparing the computed and observed discharges. The orifice discharge coefficient will be evaluated separately.

Low Weir Discharge Coefficients.—When both sides of the discharge equation for the low weir are divided by $\sqrt{2g}L_L H^{3/2}$, the result is

$$\frac{Q}{\sqrt{2g}L_L H^{3/2}} = C_{RL}\frac{2}{3} + C_{TL}\frac{8}{15}Z_L\frac{H}{L_L} \qquad (21)$$

If the term on the left is plotted against H/L_L, then the resulting relationship is linear. The intercept at $H/L_L = 0$ will permit evaluation of C_{RL}. From the slope of the line C_{TL} can be evaluated. Values of Q and H obtained during the low weir tests listed in table 2 were used to evaluate C_{RL} and C_{TL}.

The data for the low weir have been used to compute $Q/\sqrt{2g}L_L H^{3/2}$ and H/L_L. They are plotted in figure 42. Because the data points for the two lowest values of H/L_L are in poor agreement with the rest of the data, they were neglected in the subsequent analysis.

The curve shown in figure 42 was fitted by least squares. Dash lines have been drawn 1 percent above and below the least squares curve to show how well the curve fits the data. The correlation coefficient is 0.975. The intercept is 0.3721, from which

$$C_{RL} = 0.558 \qquad (22)$$

and the slope is 0.3136, from which

$$\frac{8}{15}C_{TL}Z_L = 0.3136 \qquad (23)$$

and, since $Z_L = 0.789$,

$$C_{TL} = 0.745 \qquad (24)$$

As noted previously, it is assumed that the values of C_{RL} and C_{TL} determined for the low trapezoidal weir are valid for the high trapezoidal weir. In equation form

$$C_{RH} = C_{RL} = 0.558 \qquad (25)$$

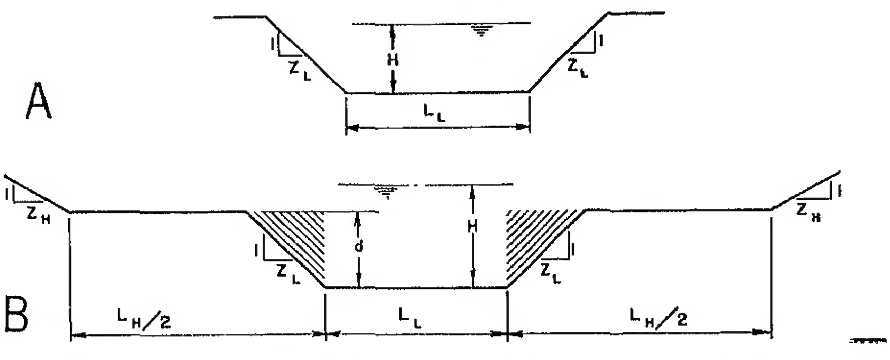


Figure 41.—The trapezoidal weir: A, Low weir; B, compound weir.

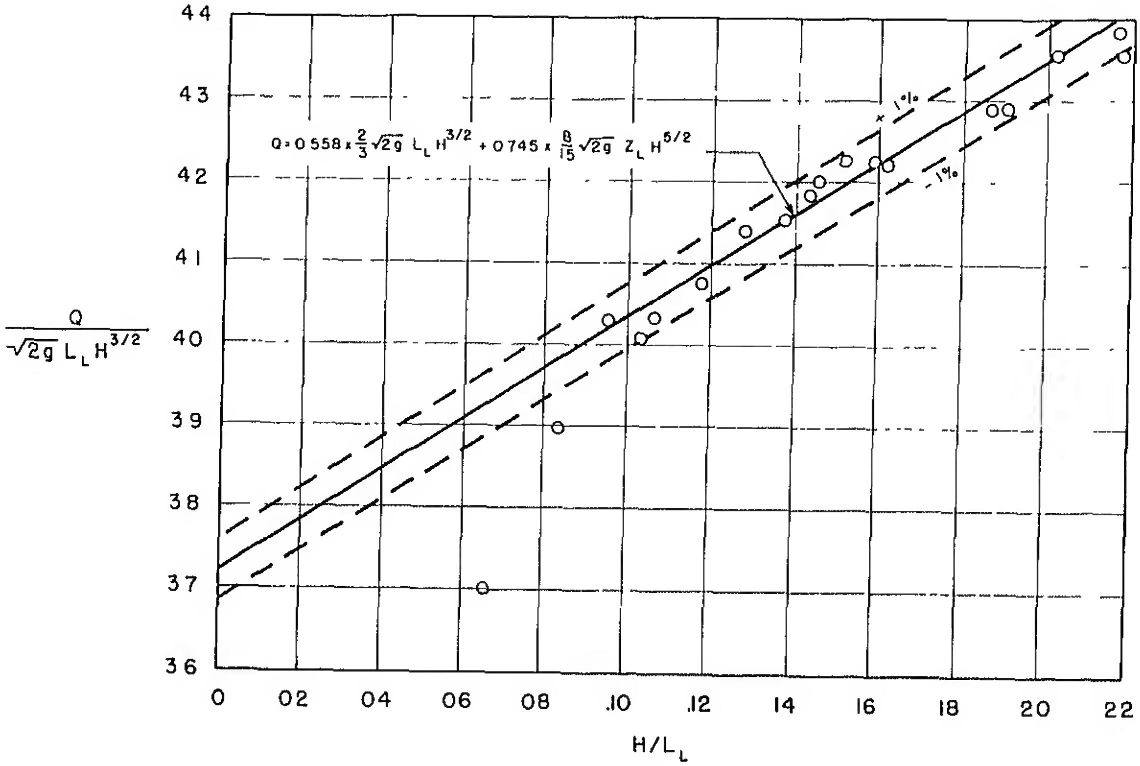


Figure 42.—Dimensionless head-discharge relationship for low trapezoidal weir.

and

$$C_{TH} = C_{TL} = 0.745 \qquad (26)$$

Orifice Discharge Coefficient.—The head on an orifice is ordinarily measured to the center of the orifice. Here, however, the head is measured to the bottom of the orifice, so a head correction is required. Furthermore, the orifice under consideration is operating under a low head and there is a variation of the velocity along the vertical, so a submergence correction must be applied. These two corrections are combined in the following derivation by Clayton L. Anderson.

Figure 43 is a sketch of the triangular orifice. The increment of discharge dQ_{TO} is the product of the elementary area $dA = 2Z_L\, x\, dx$ and the velocity through the area $V = \sqrt{2g(H-x)}$ or

$$dQ_{TO} = 2\sqrt{2g}\,Z_L\, x\,\sqrt{H-x}\, dx \qquad (27)$$

The total discharge Q_{TO} through the triangular orifice is

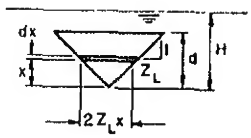


Figure 43.—Triangular orifice with head measured from bottom and under low submergence.

$$Q_{TO} = 2\sqrt{2g}\,Z_L \int_0^d x\,(H-x)^{1/2}\,dx \qquad (28)$$

$$= 2\sqrt{2g}\,Z_L \left[-\frac{2}{15}(2H+3x)(H-x)^{3/2}\right]_0^d \qquad (29)$$

$$= \frac{4}{15}\sqrt{2g}\,Z_L \left[2H^{5/2} - (2H+3d)(H-d)^{3/2}\right]$$

$$(30)$$

To make the expression in brackets dimensionless, the numerator and denominator are multiplied by $d^{5/2}$, and to put the equation in the form of the orifice equation, the numerator and denominator are multiplied by $2\sqrt{H/d}$.

When this is done,

$$Q_{TO} = \frac{8}{15}\sqrt{2g}\,Z_L d^2 H^{1/2}\left\{\frac{1}{2\sqrt{H/d}}\left[2\left(\frac{H}{d}\right)^{5/2} - \left(2\frac{H}{d}+3\right)\left(\frac{H}{d}-1\right)^{3/2}\right]\right\} \qquad (31)$$

If the term in braces—the correction for measuring the head from the bottom of the orifice and the correction for low head—is called the head-submergence coefficient C_{HS} then

$$C_{HS} = \frac{1}{2\sqrt{H/d}}\left[2\left(\frac{H}{d}\right)^{5/2} - \left(2\frac{H}{d}+3\right)\left(\frac{H}{d}-1\right)^{3/2}\right] \qquad (32)$$

In addition to this coefficient, the theoretical equation for the discharge of the triangular orifice must be modified by an orifice discharge coefficient C_{TO}. When these coefficients are introduced, the orifice discharge equation becomes

$$Q_{TO} = C_{TO}C_{HS}\frac{8}{15}\sqrt{2g}\,Z_L d^2 H^{1/2} \qquad (33)$$

To evaluate C_{TO}, this equation is compared with the triangular weir equation. If in equation 33 $H = d$, then the orifice is functioning as a triangular weir; and since $C_{HS} = 1$ when $H = d$, the equation becomes

$$Q_{TO} = C_{TO}\frac{8}{15}\sqrt{2g}\,Z_L H^{5/2} \qquad (34)$$

Comparing equation 34 with the triangular weir portion of equation 19 shows them to be identical in form. Furthermore, since the discharges given by both equations must be identical when $H = d$, it follows that

$$C_{TO} = C_{TL} = 0.745 \qquad (35)$$

Since

$$C_O = C_{TO}C_{HS} \qquad (36)$$

and both C_{TO} and C_{HS} have been evaluated, C_O has also been evaluated.

Agreement With the Observed Data

Now that the head-discharge equations have been developed and the coefficients have been evaluated, it is possible to compute the discharge for each experimental head and compare it with the observed discharge.

The discharges for the low trapezoidal weir were computed from the equation

$$Q = 0.558 \times \frac{2}{3}\sqrt{2g}\,L_L H^{3/2} + 0.745 \times \frac{8}{15}\sqrt{2g}\,Z_L H^{5/2} \qquad (37)$$

and the discharges for the compound trapezoidal weir from the equation

$$Q = 0.558 \times \frac{2}{3}\sqrt{2g}\,L_L H^{3/2} + 0.745\,C_{HS}\frac{8}{15}\sqrt{2g}\,Z_L d^2 H^{1/2} + 0.558 \times \frac{2}{3}\sqrt{2g}\,L_H (H-d)^{3/2} + 0.745 \times \frac{8}{15}Z_H\sqrt{2g}(H-d)^{5/2} \qquad (38)$$

where L_L = 41.80 feet, L_H = 70.42 feet, d = 9.07 feet, Z_L = 0.789, and Z_H = 2.99.

The computed discharges Q_C are listed in table 2. The percentage difference from the observed discharge is given in the last column.

For channel flow, it is apparent that the equation well represents the data. Only for the two lowest discharges are the differences greater than 1.6 percent.

For overbank flow, the differences vary from − 1.1 to + 5.7 percent. This agreement is good; in fact, the agreement is excellent considering that the assumptions made in developing the equations for the compound trapezoidal weir and evaluating its coefficients are not precise.

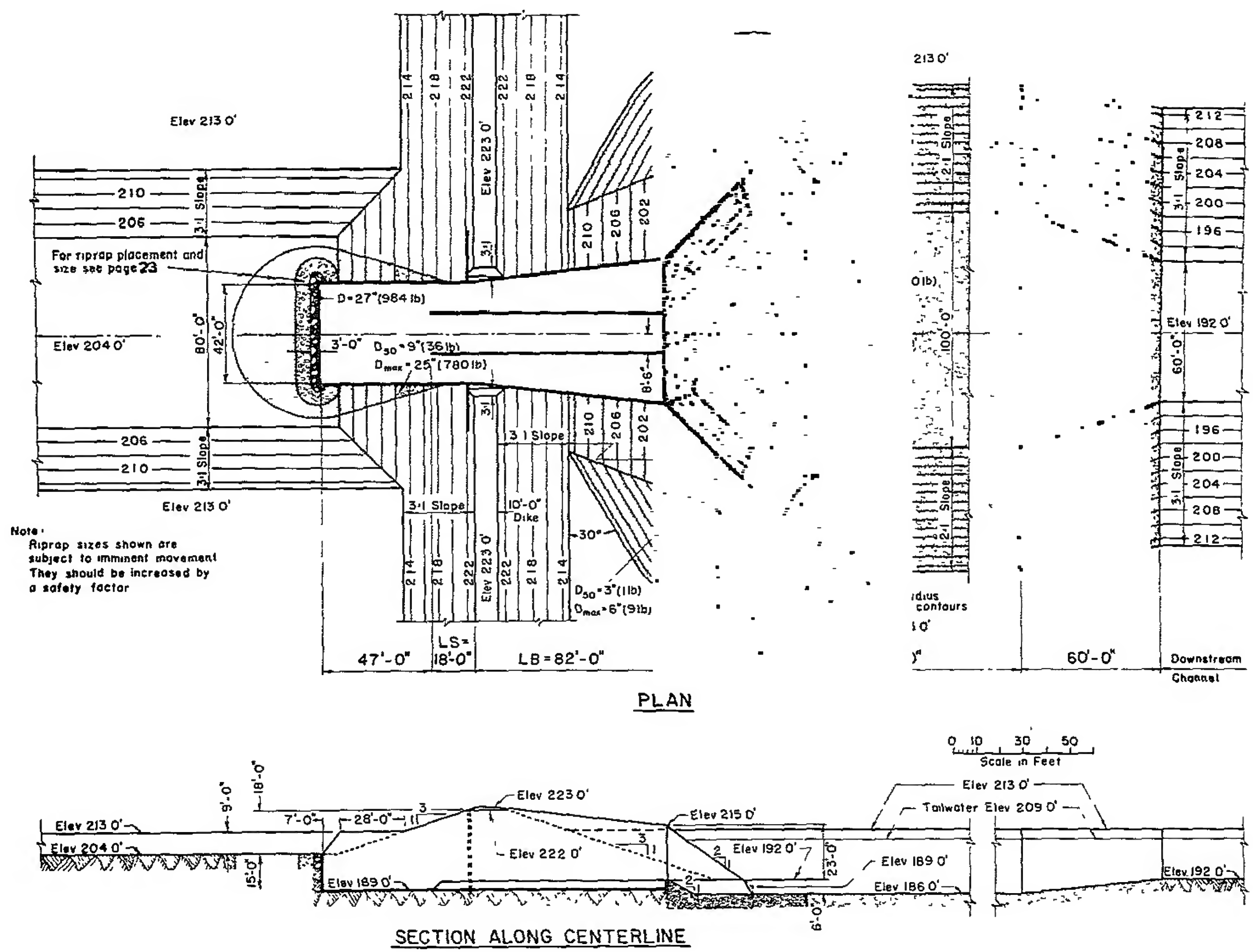


Figure 44.—Recommended design.

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

The model tests have produced an experimentally developed design for the box inlet drop spillway and for the protection required in the upstream and downstream channels. Some of the results obtained will have general application.

Approach Channel

There will be excessive scour of the approach channel unless riprap is used to protect the bed in the vicinity of the box inlet drop spillway.

It is recommended that the riprap close to the box inlet crest be placed as shown in figure 44 but that its size be increased by a safety factor.

It is recommended that the riprap placement farther upstream and its size, increased by a safety factor, be designed according to the procedures in "Approach Channel Velocity Distribution," page 16; and "Design of Rip-

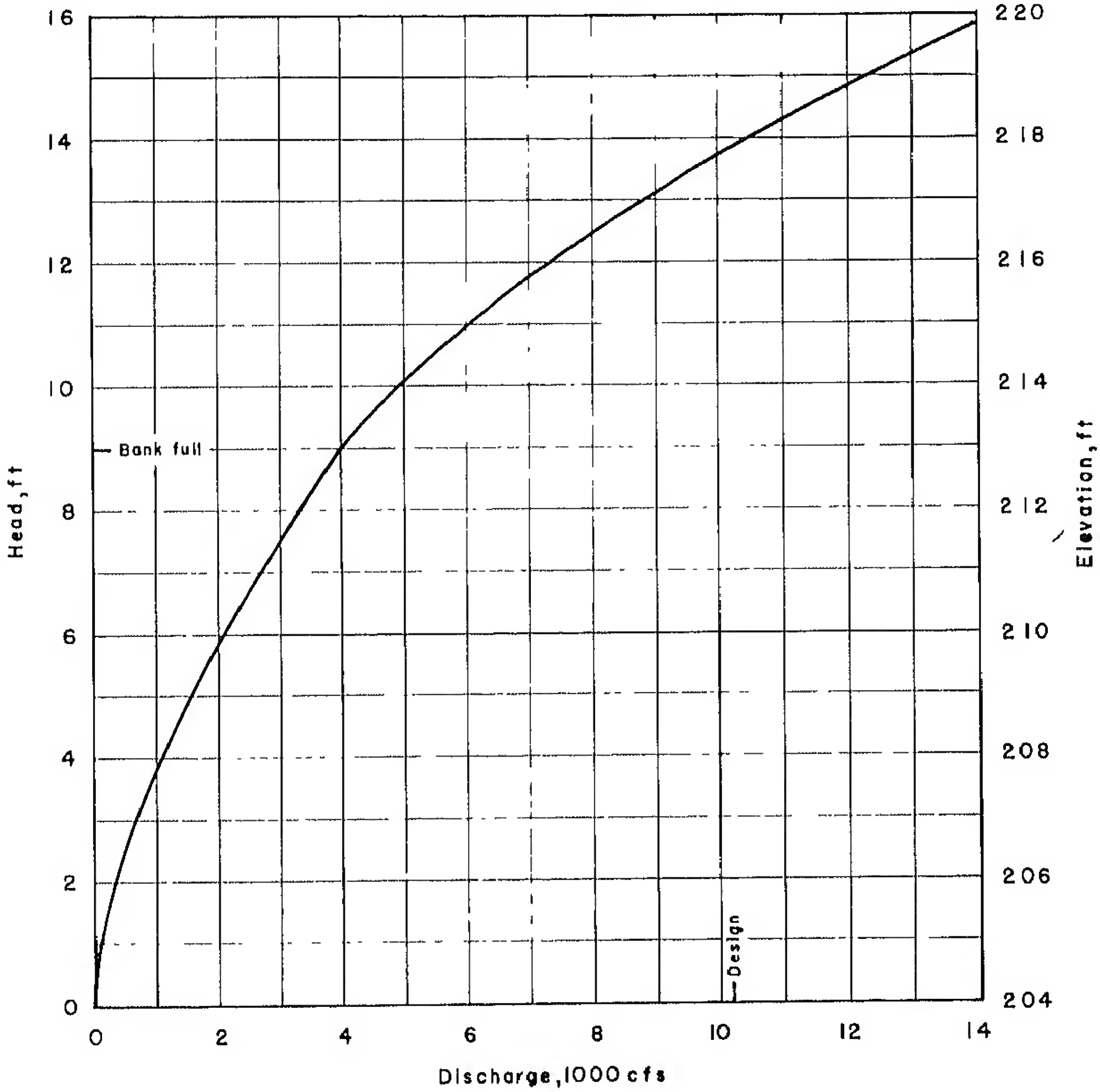


Figure 45.—Head-discharge relationship for recommended design.

rap," page 22, or equations 13 to 16, page 23.

Downstream Channel

The downstream channel could have been stabilized with small riprap if it had been shaped to fit the self-formed shape shown in figure 25.

If is recommended that the riprap sizes shown in figure 44, increased by a safety factor, be placed as shown in figure 44. The dimensions shown in figure 44 were obtained from figure 32.

Trapezoidal Weir Proportions

The dimensions of the compound trapezoidal weir can be determined from equation 19, page 44, for the low weir and equation 20, page 45, for the compound weir. The coefficients for use in these equations are given in equations 25, 26, 36, 35, and 32, pages 45, 46, and 47.

The recommended weir dimensions are given in figure 44, and the head-discharge relationship is given in figure 45.

Sweepout

The stilling basin will perform satisfactorily even if the tailwater falls below the design level. It is nevertheless recommended that the tailwater not be intentionally reduced below the design level.